

FLOOD AND COASTAL EROSION RISKS AND COST BENEFIT ANALYSIS FOR LEGUAN SOLAR FARM SITE REPORT (WORK PACKAGES 1, 2 AND 3)



NOVEMBER 2022

Submitted To:



IDB
1350 New York Avenue, NW
Washington DC, 20577

Submitted By:



CEAC Solutions Company Limited
20 Windsor Avenue
Kingston 5
Jamaica

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	Review	Submission to client	Submission to client
Prepared by:	DS/JE/MA	DS/JE/MA	DS/JE/MA
Reviewed by:	CB/OP	CB/OP	CB/OP
Approved by:	CB	CB	CB
Date:	October 28 th , 2022	October 31 st , 2022	November 11 th , 2022
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Executive summary

The Government of Guyana (GoG) under the Energy Matrix Diversification and Institutional Strengthening of the Department of Energy (EMISDE) program financed by the Inter-American Development Bank aims to improve the Guyana's energy sector. This report focused on the hazards related to and the feasibility of constructing a 0.6 MWp photovoltaic farm on the island of Leguan. Due to the site's proximity to the Leguan shoreline, at the mouth of the Essequibo River, there are concerns of the impacts that fluvial, coastal actions (storm surge and erosion) and sea level rise would have on its viability.

Hazard assessments were done after an extensive data collection programme. Data collection included topographic, bathymetric and anecdotal surveys, as well as sediment collection, precipitation, wind, wave and tidal raw data collection and analyses. The data was then corroborated from a myriad of sources including supplementary data from the client, local contacts, and further desktop studies. Analysis of meteorological, hydrology and damage information suggest that: i) coastal (and inland) stations are similar to each other, ii) flooding is associated with extreme long-duration (i.e. 1 month) rainfall events of 50-year RP or greater and iii) rainfall extreme did not have significant increasing trends or climate connections.

The hazard assessments considered fluvial flooding, sea level rise, wave climate, and coastal erosion impacts. Flood plain modeling of the 5 to 250-year return period events indicated that the site was susceptible to flooding with inundation depths ranging from 0.2 to 1.1m. The key underlying assumption is that the kokers are operational and adequate. Failure of the kokers on the island could result in considerable flooding. Prolonged monthly precipitation and tidal variation were observed to have the most significant impact. The recommended approach is to set the minimum equipment base levels at +2.08m (above MSL).

Long term accretion is approximately 2.2m per year on the site between 2013 and 2022. Notwithstanding, there is an underlying erosion rate of 1.7 m per year in the generalized area of which 0.19 m/year or 11 % is estimated to be due to SLR. The shoreline is currently stable, and as such it is not expect to accrete any further. It is expected that in the future the main driving factors of erosion along the beach area will be due to sea level rise and short-term high wave events that can cause up to 20m of erosion in high return period storms.

Several scenarios were considered ranging from an unmitigated strategy to comprehensive mitigation strategies. In summary:

- Without mitigation, the project has an IRR of 8.6%
- implementing the flood mitigation strategy results in an IRR of 8.4% and implementing the erosion mitigation strategy produced an IRR of 1.6%.
- The comprehensive mitigation of combining both flood and erosion protection lowered the IRR to 1.8%.

The flood mitigation strategy (flood risks only) is the recommended approach and proposes to raise the assets using stilts/posts for the PV panels and plinths for the BESS and transformer to a minimum recommended level of +2.1 meter above Mean Sea Level (see Table E-1)

Table E-1: Summary of financial analysis for Solar Farm with recommended flood mitigation scenario

Costs (USD)	Flood Mitigation Scenario
Cost/Investment [solar farm + flooding mitigation]	\$1,928,367
Average Annual Losses - mitigated (USD)	\$1,571
Savings from RE (USD)	\$204,311
Net Benefits (USD)	\$202,740
Net Present Value: Benefits (USD)	\$2,988,562
Internal rate of return	8.4%
Benefit Cost Ratio (BCR)	1.5

It is further recommended that an assessment of the condition and upgrading of the kokers and flood control and drainage infrastructure of the Leguan be undertaken. Additionally, the implementation of the shoreline protection works is a reasonable precondition to the implementation of the solar farm

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ACRONYM LIST

BOB	-	Back of the Beach
BOD	-	Biological Oxygen Demand
CFL	-	Courant–Friedrichs–Levy
COD	-	The chemical oxygen demand
DAC	-	The Development Assistance Centre
EBP	-	Equilibrium beach profile
EPA	-	The United States Environment Protection Agency
FOB	-	Beach Face
GIS	-	Geographic Information System
GMSL	-	Global Mean Sea Level
GNSS	-	Global Navigation Satellite System
GPS	-	Global Positioning System
HVAC	-	Heating, ventilation, and air conditioning
IPCC	-	Intergovernmental Panel on Climate Change
LED	-	Light Emitting Diode
Lidar	-	Light Detection and Ranging
MEP	-	Mechanical, Electrical, and Plumbing
MSL	-	Mean Sea Level
WWTP	-	Waste Water Treatment Plant
MWp	-	Mega Watt Peak
SLR	-	Sea Level Rise

1 Introduction

1.1 Background

The Government of Guyana (GoG) under the Energy Matrix Diversification and Institutional Strengthening of the Department of Energy (EMISDE) program financed by the Inter-American Development Bank aims to improve the Guyana's energy sector. This is through the implementation of:

1. Photovoltaic Farms in order to diversify energy sources.
2. Improvement and reinforcement of transmission infrastructure.
3. Strengthening of the Department of Energy's regulatory framework.

The project of focus for this report is the development of a grid-connected utility-scale solar photovoltaic (PV) system with a total installed capacity of 0.6 MWp and a storage capacity of 0.6 MWh located on the island of Leguan. This report will discuss the preliminary findings of assessments of possible hazards to the proposed 1.5 HA site and installed infrastructure.

The Island of Leguan is approximately 14km long and 3.2km wide and is situated in the Delta of the Essequibo River on the coast of Guyana. The estimated population is 2,500, with citizens residing in 36 demarcated villages. The typical focus of activities includes rice farming and cattle rearing. The island has three main paved roads, two of which run along the north and south coasts. The third bisects the land mass and acts as a connector of the aforementioned.



Figure 1.1 Site Location Map of Leguan PV Solar Farm.

1.2 Solar Farm Site

1.2.1 Description

The solar farm is estimated to be 40-60m from the shoreline of the Essequibo River and will occupy 2,900m² of coastal land. Elevations within the footprint of the project area were determined using topographic data collected through photogrammetry and traditional survey methods. The findings indicated that these site elevations within the footprint were within a range of 0.4-2.0m AMSL. Both the low-lying nature of the site and proximity to the river water line implies susceptibility to inundation and even erosion.

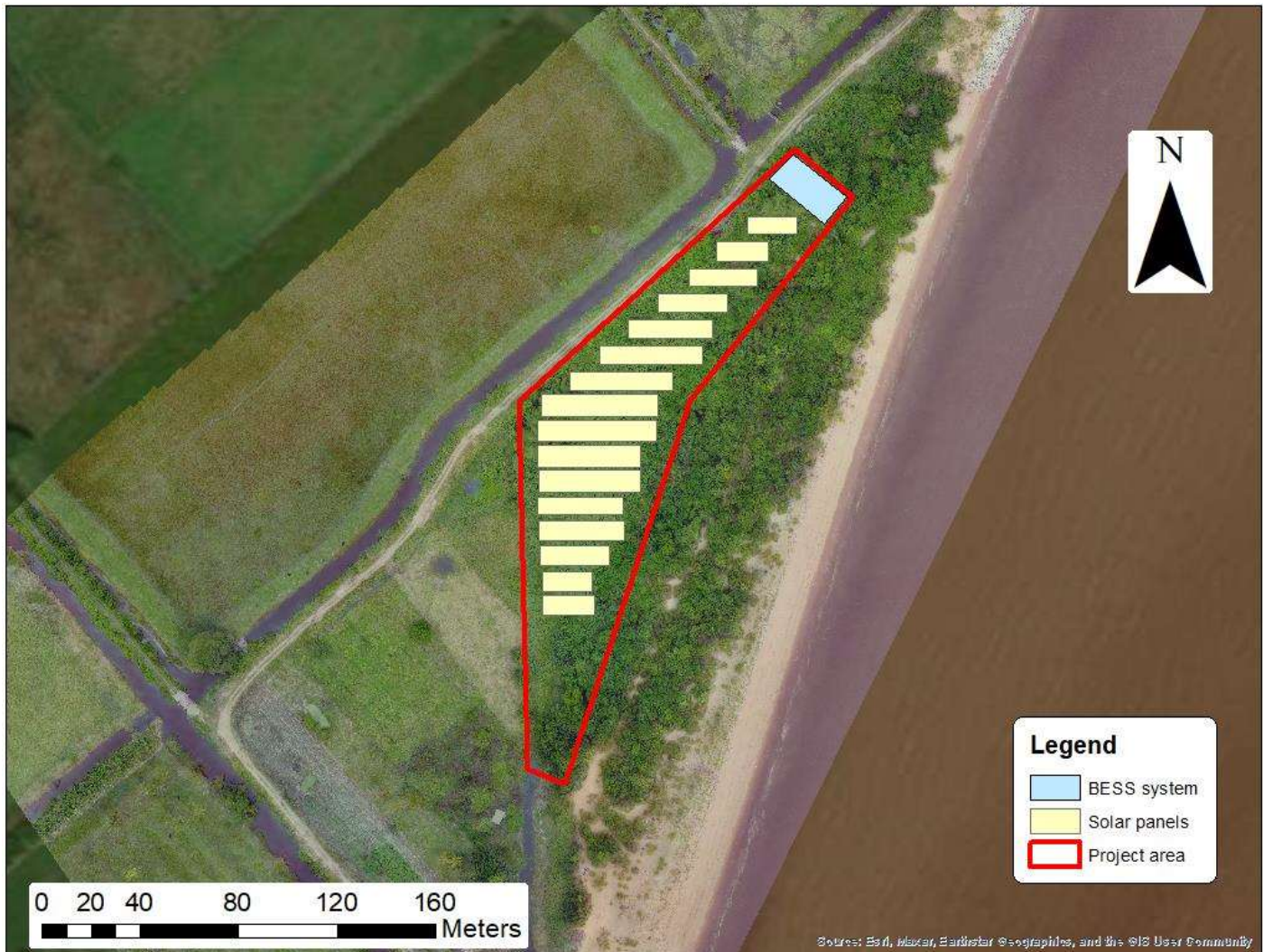


Figure 1.2 Aerial oblique of proposed Leguan Solar Farm Site Location.

1.2.2 Cost of Solar Farm

As presented by the Guyana Energy Agency (GEA), it is assumed that the solar panels will be 0.9m above the ground and the Battery Energy Storage System (BESS) will be placed on plinths approximately 0.3m above ground level. It is also assumed that the BESS will be placed approximately 0.3m off the ground. Without the mitigation measures, all assets remain susceptible to flood damage to be assessed further in the report. To obtain the cost of construction, the assets were broken up into 2 main systems:

1. The PV system
2. The BESS

The implementation of both systems was assigned by proportioning the total cost for the solar farm between the PV system and the BESS. The total cost for the project is seen in Table 2.1 while the distributed costs for the 2 systems are in Table 7.2.

Table 1.1: Cost breakdown for solar farm

#	Description	Unit cost	% CAPEX	Total cost (USD)
1	Solar PV System	536 \$/kWp	16%	321,300
2	Battery bank	950 \$/kWh	29%	570,000
3	AC Balance of Plant & Controls		4%	81,320
4	Electrical Interconnection		21%	275,316
5	Civil Works		8%	155,400
6	Transport and Installation		6%	121,714
7	Soft Cost		10%	175,381
8	Contingency & Other		5%	85,022
	TOTAL EPC cost	2,976 \$/kWp		1,785,452

Table 1.2: Distributed cost of the solar farm between PV panels and BESS

	Implementation	Distributed costs (USD)
1	PV Panels	\$589,815
2	BESS + Inverter+ controller	\$1,195,637

1.3 Vulnerability

The solar farm consists of two (2) main operational systems. (1) A series of photovoltaic (PV) panels and (2) a battery energy storage system (BESS) which contains batteries, inverters and transformers. The majority of associated components are electrically operated and as such are highly vulnerable to prolonged submersion in water and consequently rises in flood levels caused by the Essequibo river.

In the case of the photovoltaic panels, though water resistant, the possibility exists that the assets could be rendered unusable and irreversibly damaged in the event of partial submersion. This may be caused by water penetration into the electrical encasement. As such water levels exceeding the lowest point of the panel may be detrimental. The panels were assumed to be mounted on 0.9m high elevated frames. In the case of the BESS System, it was assumed that the components are housed within an intermodal container 0.45m above its base. Due to the conductive and interconnected nature of the BESS system, it was determined that waters exceeding that level could cause irreparable damage to the entire storage system including its inverter, batteries, and transformer. See the damageability curve in Figure 1.3

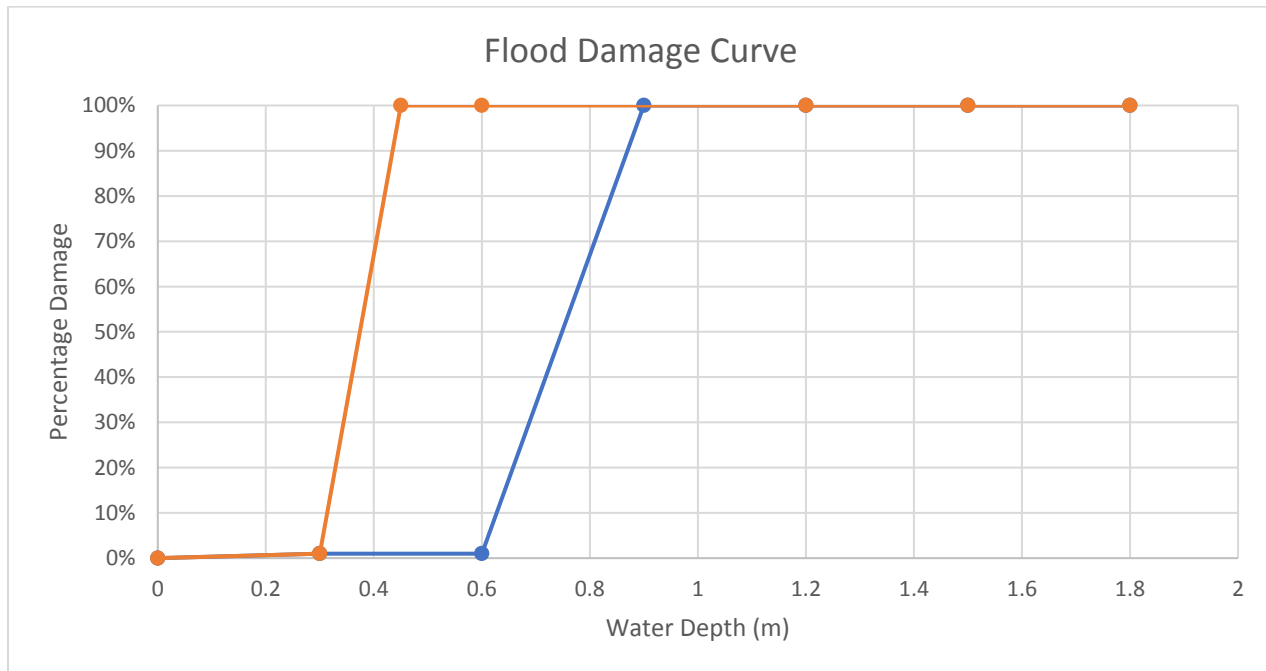


Figure 1.3 Flood damageability curve based on flood levels for Photovoltaic and Energy Storage vulnerability.

1.4 Approved Scope of Work

The scope of works for this project aims to provide supporting studies for flood risk and coastal erosion along with the CBA, to ensure mitigation measures appropriately consider all factors likely to impact the proposed Solar farm on the Leguan Island. The proposed site investigations for the Leguan Solar Photovoltaic Farm Project Site will consist of three (3) work packages as follows:

- **Work Package 1:** A Flood Risk Report for the Project Site
- **Work Package 2:** Coastal Erosion Risk Study for the Project Site
- **Work Package 3:** Cost-Benefit analysis that also integrates flood (Work Package 1) and coastal erosion risk (Work Package 2)

1.4.1 Work Package 1: A Flood Risk Report for the Project Site

The objective of this report is to understand the current and future flood risk scenarios, as well as evaluate the current design and possible mitigation measures implemented in those scenarios that may be justified from a disaster risk reduction and climate change adaptation perspective derived from quantitative risk analysis. The consultant's responsibilities include the provision of the following:

- i. Flood risk analysis report.
- ii. Drawings or georeferenced layers with the 5, 10, 25, 100 and 250-years return period flood footprint, including water depths, WSE, and velocities.
- iii. Simulation reports are performed by modelling studies of both current and future scenarios.
- iv. Recommendations to reduce and manage flood risk through both structural (use of Nature-Based Solutions encouraged) and non-structural measures.
- v. Cost-benefit analysis (CBA) integrating flood risk in the current scenario (with and without the site built as per current designs) and in future scenarios (site built as per the current design and climate change projections included) and with the different proposed measures to assess its efficacy and efficiency.
- vi. Prioritization of measures based on both CBA and environmental impact considerations.

1.4.2 Work Package 2: A Coastal Erosion Risk Study for the Project Site

The goal of the coastal erosion risk study for the site is to determine the current and future scenarios of potential coastline regression/ erosion risk including the effects of climate change driving to Sea Level Rise and variation in water stream patterns. The consultant's responsibilities include, but are not limited to the provision of the following:

- i. Study of coastal transport capacity.
- ii. Sedimentary balance and evolution of the coastline focusing on coastal potential erosion and regression including dynamics resulting from the effects of climate change.
- iii. Recommendations to reduce and manage flood risk through both structural (use of Nature Based Solutions encouraged) and non-structural measures. The latter may include, for example, a monitoring plan of the planned structural actions, beach nourishment plan if material available, etc.
- iv. Cost-benefit analysis (CBA) integrating flood risk in the current scenario (with and without the site built as per current designs) and in future scenarios without (site built as per the current design and climate change projections included) and with the different proposed measures to assess its efficacy and efficiency.

2 Extreme Flood Events and Background Erosion

2.1 Flood Events and Damage History

Flood events experienced in Guyana usually occur during the mid-summer or later rainy season. Guyana experiences its wet season between May and early August as well as from December to January. There is an underlying trend of more extreme depths (Figure 2.1). A summary of extreme rainfall events in ranked order can be seen in Table 2.1.

Monthly Climatology of Min-Temperature, Mean-Temperature, Max-Temperature & Precipitation 1991-2020
Guyana

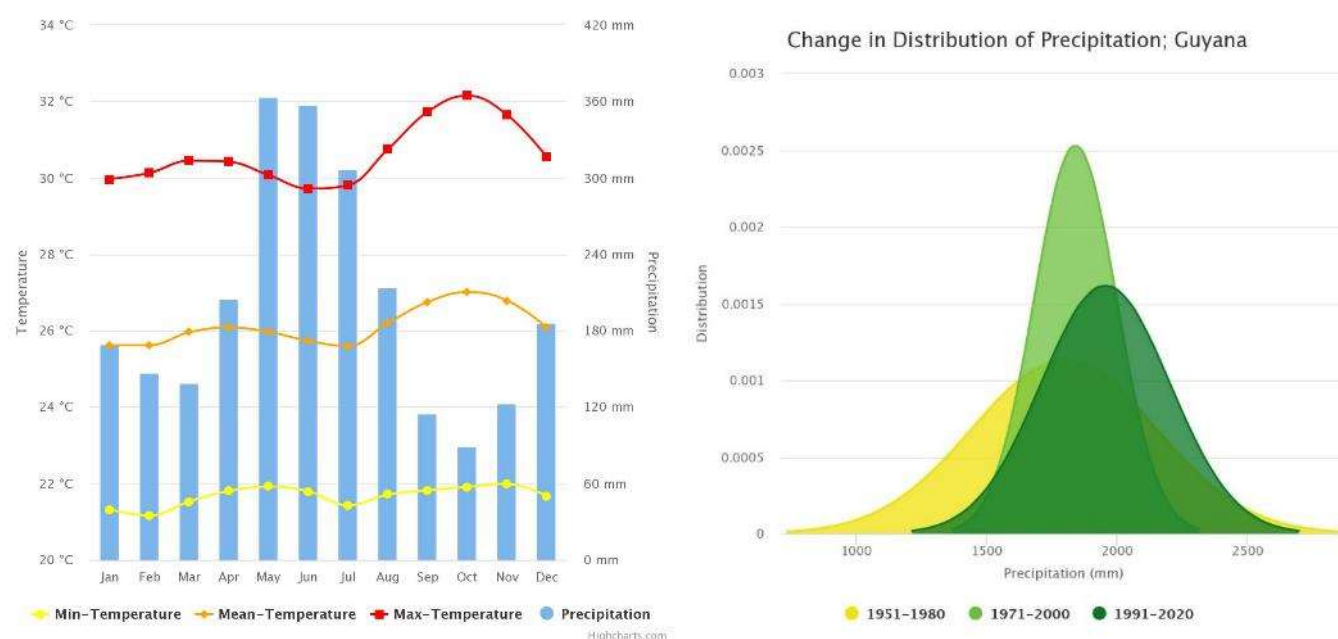


Figure 2.1. Rainfall climatology (left) and distribution (right) for Guyana, from World Bank¹

Table 2.1: Ranking of Daily Maximum Rainfall, Monthly Maximum Rainfall and Annual Rainfall in Georgetown (1886-2016)²

Table R 1.3.1 Ranking of Daily Maximum Rainfall, Monthly Maximum Rainfall and Annual Rainfall in Georgetown (1886-2016)

Ranking	Daily Maximum Rainfall		Monthly Maximum Rainfall		Annual Rainfall	
	mm/day	Date	mm/month	Month/Year	mm/year	Year
1	245.1	14 June, 1892	1108.2	January 2005	3749.3	1954
2	210.2	15 July, 2015	1022.0	December 1942	3454.3	1892
3	194.3	1 December, 1936	919.6	December 2008	3434.7	1893
4	192.8	8 May, 1951	831.0	January 1950	3365.2	2008
5	186.0	19 November, 2014	824.5	December 1949	3315.5	2005
6	181.9	7 January, 1934	822.7	December 1891	3137.9	1889
7	174.8	26 December, 1893	822.2	June 1892	3122.0	1898
8	174.0	1 November, 1974	716.8	December 1973	3097.4	1943
9	166.1	16 January, 2005	711.1	January 1934	3076.6	1956
10	160.0	14 May, 1945	688.5	December 1933	3022.5	1938

Source: HS-MOA (Rainfall over the past 15 years)

¹ World Bank Group. (n.d.) Climatolgy. <https://climateknowledgeportal.worldbank.org/country/guyana/climate-data-historical>

² National Drainage And Irrigation Authority. (2017). Data Collection Survey On Drainage Capacity In Georgetown In The Co-Operative Republic Of Guyana Final Report. [12292934.pdf \(jica.go.jp\)](https://www.jica.go.jp/12292934.pdf)

2.1.1 1942 event

In the year 1942, Guyana experienced one of its most extreme rainfall events. As seen in Table 2.1, 1022mm of rainfall was recorded in December by the rain gauge station in the Botanical Gardens of Georgetown. Details on the extent of damages experienced at this time were somewhat scarce, however the acknowledge of this event bears some significance due to the similarity in terms of precipitation data when compared to floods that occurred in 2005. Comparison of the two events using the Georgetown and Mazaruni stations saw variations of less than 8%.

Based on this observation that likelihood that damages or flood levels of a similar nature occurred are likely. Several sources including reports from The World Bank in Latin American and The Caribbean (2010)³, indicated that rainfall events of this magnitude were typically associated with historical instances of agricultural loss. The damages that result from these events are mainly felt in the rice industry and other non-traditional farming practices, valuing at over USD\$4,752,000.00. Among vegetable farmers in Leguan, there is a shared sentiment that significant losses in their practice is from the annual episodes of flooding on the island.

2.1.2 1971 event

A review of the International Disaster Database (EM-DAT) indicated that In July of 1971 damages of up to USD\$1,338,000 in that year were incurred in the areas of Cane Grove and East Cost to flooding. In total, 21,000 locals were affected by the flood despite relief response from the Office of U.S. Foreign Disaster Assistance (OFDA). The percentage of economic loss to GDP was 0.1% from the event. Cross referencing of precipitation data for this time period indicated there was an average of 270-453mm at the Georgetown and Timeheri Stations for the month of occurrence. Return period analyses done in section 3.7 indicate that this even had less than a 2.5 Yr. recurrence interval. Based on this it is likely that flooding may have been as result of infrastructure and drainage failure.

2.1.3 2005 event

In January 2005, intense flooding was experienced within the northern coastal regions of Guyana. These included the West Demerara, the Essequibo Islands, Demerara and the West Mahaica-Berbice Area. The World Bank's: Guyana Preliminary Damage and Needs Assessment (2005) indicates that these areas comprised 75% of Guyana's population inclusive of the capital, Georgetown, whereas the EM-DAT International Disaster Database indicated 275,000 persons were affected.

During the period starting from December 24th to the beginning of February 2005, catchments contributing to the coastal river outfalls in the northern regions of Guyana saw precipitation over a month that was sustained and higher than normal in intensity. Precipitation data indicated that 1540mm of rainfall was observed in a 6-week period. This is well above the 182mm typically observed in the month of January, as indicated by an analysis of precipitation data on the Essequibo Islands between 2011 -2021 and corroborated by the World Bank's assessment subsequent to the flooding event in 2005. There was also a spike over a 5-day period, in mid-January which included approximately 650mm of precipitation⁴.

The increased precipitation, coupled with the onset of the increased tidal levels caused by spring tide, saw significant inundation in the northern coastal regions of Guyana. The combined scenarios saw the exceedance of capacities in conservancies and the limiting of seaward outflows due to failing infrastructure such as heavily silted drains, failing gates and pumps. Depths of 1.27m and higher were observed in some regions with EMDAT estimating total adjusted damage of US\$645,332,000. In some accounts, flood depths took up to 3 weeks to subside.

2.1.4 2015 event

In July 2015, heavy rainfall and flooding were experienced in the coastal regions of Guyana. According to EM-DAT International Disaster Database, the event took place between July 15th and 21st. It was noted that approximately 100,000 persons were affected. The Regions outlined included Barima-Waini, Pomeroon-Supenaam, The Essequibo Islands,

³ World Bank LAC. (2010). Agricultural Insurance Component Pre-feasibility Study Report.

(<https://documents1.worldbank.org/curated/en/214781468238475145/pdf/756520ESW0P1170Guyana0Insurance0Web.pdf>)

⁴ Isabella Bovolo (2013) Managing Flood Risk in Guyana The Conservancy Adaptation Project 2008-2013

Demerara-Mahaica and Mahaica-Berbice. See Impact *Table 3.1*. The specific magnitude of flood depths was unclear in various instances how depths exceeding 0.6m of the canals were reported.

Table 2.2 Accounting impacts as per Emergency Plan of Action report generated by Guyana Red Cross Society

Region	Recorded Impacts
Barima-Waini	1. Community Flooding (unknown depth)
Pomeroon-Supenaam	1. Flooding of Pomeroon river bank 2. High Water Level in Tapacuma Water Conservancy
The Essequibo Islands	1. Flooding of lower Essequibo River Banks 2. Canals covered by 0.6m of Water
Demerara-Mahaica	1. Flooded Farmlands and “high” water level (unknown depth) 2. Heavy can siltation
Mahaica-Berbice	1. Flooded Farmlands and “high” water level (unknown depth)

Assessment of rainfall gauge data record on the island of Leguan between 14th and 18th indicated that 221mm of precipitation occurred over a 5-day period with a significant peak of 145mm on the 15th of July. Although possible, there was no account of the impact of high tide on the inundation. There were however several accounts of failing mechanical Best Management Practices (BMPs), such as sluice gates and pumps, that may have inhibited the efficient mitigation of the raising water levels.

2.1.5 2019 event

In July 2019, heavy rains were experienced within sections of Georgetown and other coastal villages in Regions 2 and 3, as well as in Devonshire Castle, Hampton Court and other nearby divisions. Stabroek News (2019), a local publisher in Guyana, reported that flooding in the areas previously listed resulted in 30% of rice planted in flood-prone areas being lost⁵. The cause of such an event was the heavy rainfall and reportedly poor drainage conditions of channels and other flood control infrastructure.

Guyanese Online (2019) echoed the views of the population, reporting that weeds were blocking the free flow of water in drains and canals. Low-lying areas such as South Ruimveldt, Festival City North Ruimveldt, Streets in East Ruimveldt and sections of the West Ruimveldt Front Road also experienced flooding (as seen in Figure 2.1). Roadways were “covered” with floodwaters ranging between 50mm and 100mm.



Figure 2.2: Flooded road in Ruimveldt (July 2019)

⁵ Stabroek News. (2019). Some Essequibo Coast Villages Still Flooded. [Some Essequibo Coast villages still flooded - Stabroek News](#)

Irene, a resident of South Ruimveldt, gave a personal account of her situation in South Ruimveldt, explaining that, “In most recent times... we haven’t seen this kind of flooding.” Another account from Harmon of North Ruimveldt says that, “I have been living here for over forty years and this is an issue. The main canal out there needs to be cleaned.” Chief City Engineer, Colvern Venture, explained that all of the drainage and irrigation pumps operated by the Mayor and City Council were operational, but that pumps at Kitty and Lilienthal were experiencing some issues that were electrical in nature.⁶ In conjunction with other accounts by the citizens, poor drainage and failed pumps are seemingly the main contributors to flooding in the area.

According to another paper, Guyana Chronicle⁷, in May 2019, reports from the Region 8 municipality suggested that 25 homes had been affected and required immediate relief. Homes below Danjou Hill and near the airstrip were also badly affected, with some homes under several feet of water (see Figure 2.4). A resident in the area told the newspaper that his home was flooded and he had lost foodstuff and electrical items in the disaster.



Figure 2.3: Flooded section of Mahdia (May, 2019)

In general, rain gauge stations in 2019 within the area recorded rainfall amounts above their long-term averages. Average amounts of rainfall recorded by stations ranged from 140.3 mm in Region 6 (over 11 days) and 336.2 mm in Region 8 (over 21 days)¹⁰. A map showing the rainfall distribution for the month of August in 2019 (Figure 2.5) indicates that Leguan area experienced at least 100mm of rain for that month.

⁶ Stabroek News. (2019). Guyana: Sections Of Georgetown Flooded After Heavy Rain On July 1, 2019. [Guyana: Sections of Georgetown flooded after heavy rain on July 1, 2019 | Guyanese Online](#)

⁷ Guyana Chronicle. (2019). Flooding In Hinterland Locations. [Flooding in hinterland locations - Guyana Chronicle](#)

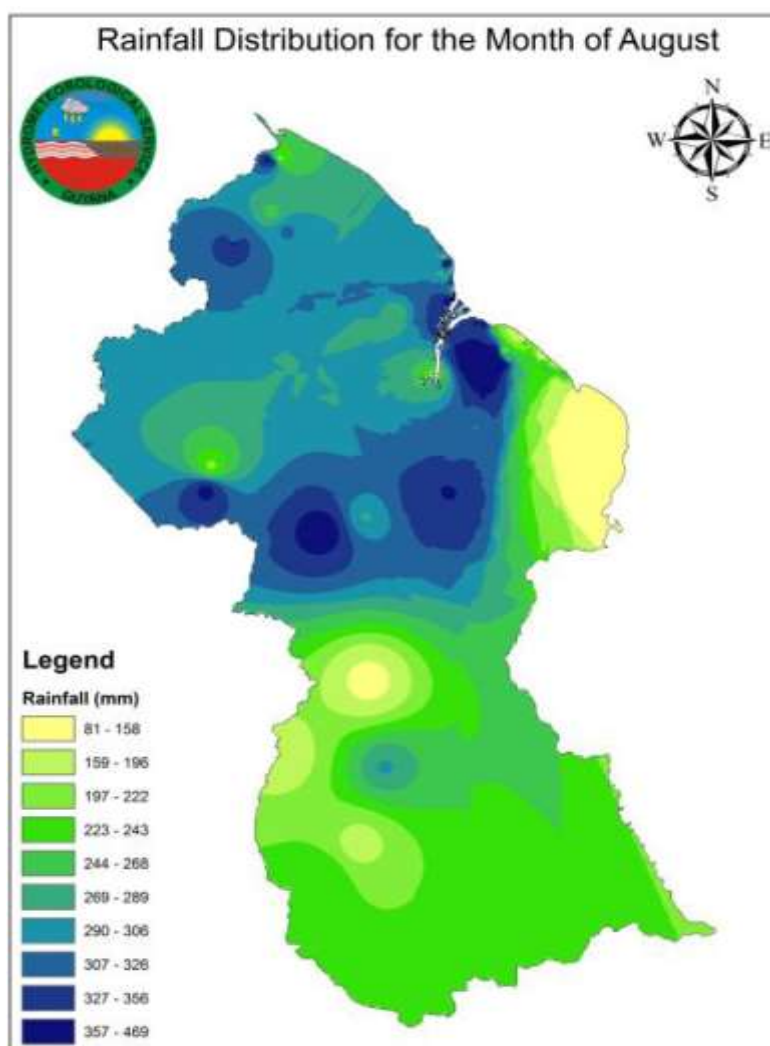


Figure 2.4: Rainfall distribution map for August, 2019 for Guyana⁸

2.1.6 Damage

While ten events are recorded in the EM-DAT database, the most significant four events suggest that damage can range from USD\$337,000 to USD\$1.3 million (2020) as a result of monthly rainfall depths of 373.9 mm to 1098 mm of rainfall (recorded in Georgetown and La Bagatelle).

Event	Rainfall (mm)	Damage (USD, 2020)
1971	448	\$1,338,000
2005	1098	\$645,332,000
2015	455.5	\$336,840.09
2019	373.9	N/A

⁸ Hydrometeorological Service of Guyana. (2019). Farmer's Monthly Weather Bulletin. [September-FMB-2019-.pdf \(cimh.edu.bb\)](#)

2.2 Erosion

2.2.1 Background Erosion Rate

Coastal erosion occurs whenever the deposition of new material is less than the removals of material from the shoreline, which may lead to the landward erosion of the beach berms and undercutting of manmade structures situated near the coastline. An assessment was conducted using satellite imagery over a 19-year period (2002-2021) of long-term erosion trends. This allowed for the identification of erosion hot spots and the long-term threats to the project area from retreating shorelines. It is important to identify these erosion hotspots that might require stabilization and make recommendations to reduce the vulnerability. Long-term assessments of the shorelines in the Essequibo River delta were conducted for 4 islands:

1. Leguan
2. Wakenaam
3. Troolie
4. Hogg

Unobstructed and clear satellite imagery was identified for as many years as available between the 2002-2021 and georeferenced to correct distortion. Visible shoreline extents were then compared to a single base line and analysed for nine (9) locations spread across the four islands. See Table 3.2 and Figure 3.1 for areas identified to be experiencing either erosion or accretion.

Table 2.3: Summary of Coastal Accretion/Erosion for the Essequibo River Delta Islands

Location Tag	Years of Observation	No. of Measurement taken	Overall Process	Avg. Rate per year (m/yr)	Island
1 (Project Area)	2002 - 2019	7	Accretion	2.2	Leguan
2	2002 - 2021	7	Erosion	4.4	
3	2007 - 2020	4	Accretion	0.7	
4	2000 - 2020	6	Erosion	1.2	Wakenaam
5	2004 - 2020	4	Erosion	0.2	
6	2015 - 2020	3	Accretion	0.4	Troolie Island
7	2006 - 2020	4	Accretion	0.4	
8	2015 - 2020	3	Erosion	0.2	Hogg Island
9	2007 - 2020	4	Accretion	0.6	
Overall trend			Erosion	1.7	

Results obtained show that though most islands are experiencing some level of accretion, the overall trend was determined to be erosion as the rate of erosion observed was typically of a greater magnitude each year. Higher rates of erosion were typically observed at the most seaward points of the island possibly due to greater exposure to more significant wave climates. When compared to the expected rates of loss due to SLR, this preliminary analysis indicates that deeper more in-depth studies are required in order to quantify the sediment budget and necessary shoreline stabilization measures.

The project site is located at the shoreline labelled “1” in Figure 2.1. Based on the collected data, the study area is accreting sand at a rate of 2.2 m/year. Comparatively, this area displayed the most accretion when compared to the other shorelines. However, it is key to note this is a long-term analysis, and that the site may be susceptible to erosion by short-term events.



Figure 2.5: Long Term Shoreline Erosion Rates for Islands at the Essequibo mouth, Guyana. Years of Observation: 2002 - 2021

2.2.2 Sea Level Rise Contribution

Sea level rise erosion/shoreline recession was estimated using the Brunn's Rule formula which estimates, the magnitude of the retreat of the shoreline of a sandy shore in response to changes in sea level. The likely shoreline recession from SLR was estimated from the geometry of the beach slope (2%) and the local Sea Level Rise of 8.5 mm/year. The underlying shoreline recession rate was calculated at 0.19 m/year which represents 11% of the overall erosion rate observed for the Essequibo islands as a collective.

Table 2.4 Brunn's Rule Results for the project area

Parameter	Profile		
	1 (0+200)	2 (0+400)	3 (0+600)
Berm Height, B (m)	3	3	3
Rate of sea level rise, S (m/yr)	0.0085	0.0085	0.0085
Offshore profile, W* (m)	100	118	125
depth of offshore limit, h* (m)	2.2	2.2	2.2
Long term erosion based on Bruun Model (m) /yr	-0.16	-0.19	-0.20
Estimated change in 25 years (m)	-4.09	-4.82	-5.11



Figure 2.6 Historical shoreline positions on project area shoreline from 2002 to 2021

2.3 Summary

Through analysis of extreme historical events, the high susceptibility to flooding on the Island of Leguan were made explicit. In the case of smaller magnitude events, like those observed in 2015 and 2019 (2-3-year RP), typical inundation depths ranged from 0.1m up to 0.6m. However, for more significant events like the 2005 (250-year RP) event, inundation depths in some instances were in excess of 1m in multiple instances. Based on the accounts it seemed a common trend or root cause of the flooding observed was not only the quantity of precipitation but also the operation of flood control infrastructure such as pumps, silted drains and failing tidal control structures. These failures naturally exacerbated some flood depths and extents. As such damages have been historically associated with a combination a significant rainfall and failing infrastructure.

Based on background erosion rate observed at the site, the threat of erosion outside of short-term events was deemed to be low. This was informed by the high accretion rate of 2.2m/year. Notwithstanding, there is an underlying erosion rate of 1.7 m/year in the generalized area of which 0.19 m/year or 11 % is estimated to be due to SLR. It should be noted that accretion can quickly become erosion with change in the shoreline uses and significant anthropogenic changes.

3 Data Collection

3.1 Topography and Bathymetry

Topographic and Bathymetric data were utilized in both the hydrologic and floodplain modelling and coastal modelling processes. Bathymetry is the underwater terrain of the sea floor and other submerged entities. It generally describes the depths relative to a datum (mean sea level). In the modelling case of the floodplain modelling, it was necessary to account for bathymetric data, as it provided a basis for analysing the fluvial behaviours as well as the impact of tidal variations from the sea inland.

For the coastal processes, bathymetric data was utilized to better understand how deep-water waves would propagate into shallow waters and affect the shoreline. In addition, a combination with topographic terrain data allowed for the assessment of storm surge levels and erosion extents on the project site.

3.1.1 Topography

3.1.1.1 Existing Topography

For the large hydrological analysis, topographic data was extracted from the Japan Aerospace Exploration Agency (JAXA). This refers to a series of digital terrain model rasters collected from satellite imagery and parcelled into 30m grids. The rasters were merged using GIS and the overall terrain model of Guyana was generated. The data was then merged with bathymetric data from General Bathymetric Chart of the Oceans (GEBCO) charts to capture riverine and ocean depths. Further topographic and bathymetry data refinements are discussed below.

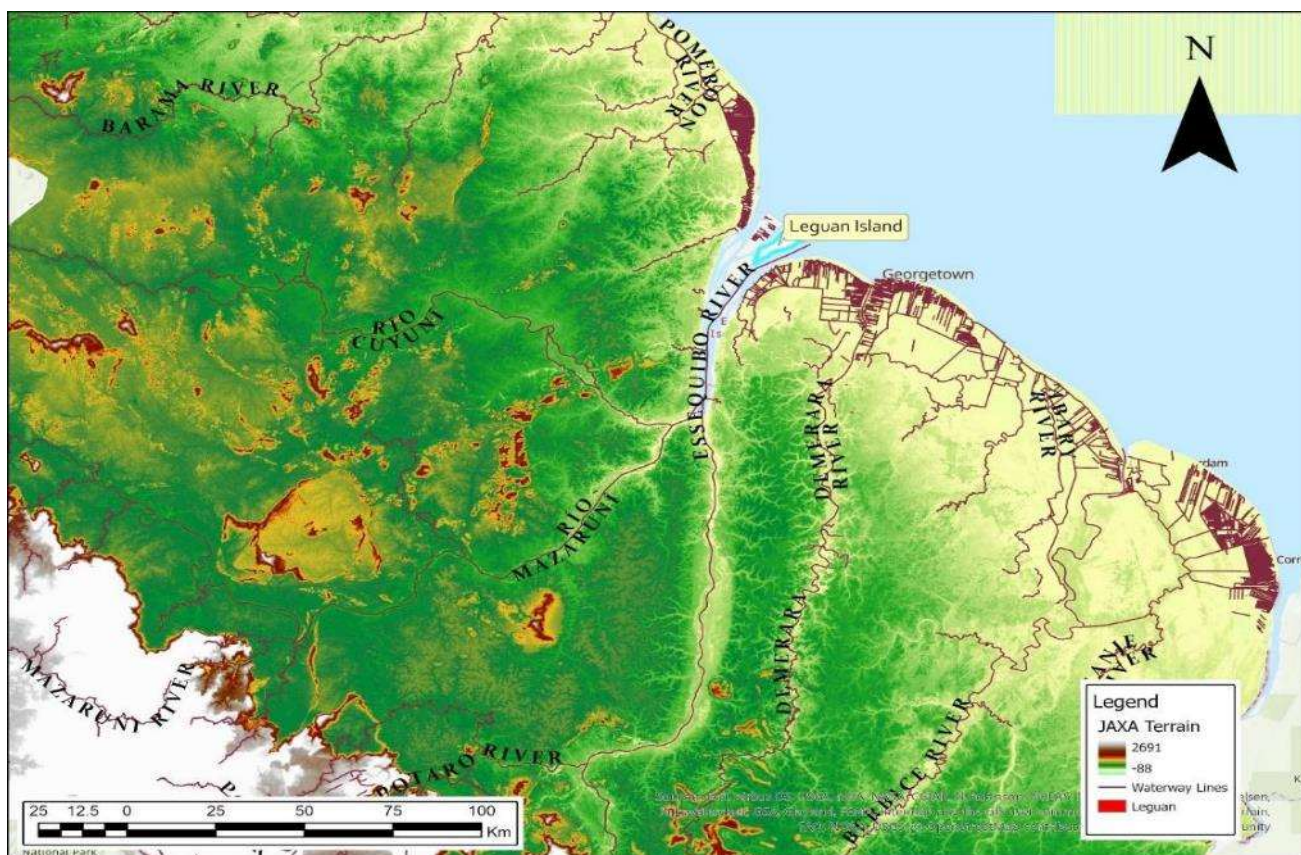


Figure 3.1 Topographic data for Guyana from JAXA.

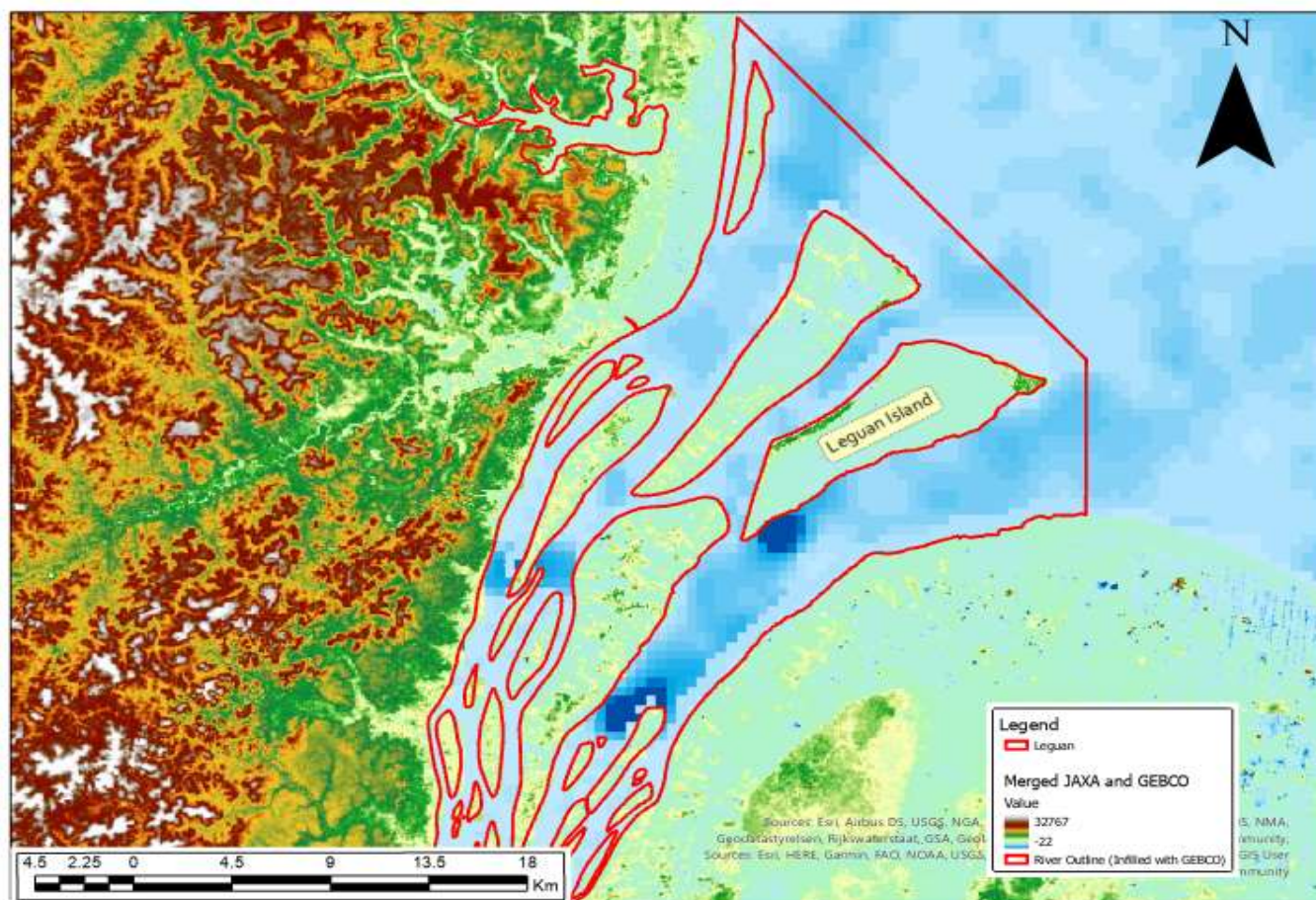


Figure 3.2 JAXA topographic 30m grid data merge with GEBCO bathymetric data

3.1.2 Photogrammetric Survey (2022)

An aerial survey was performed to generate site-specific topography in October 2022. This was done using the combination of a series of ground control points (GCPs) and captured point cloud data from a series of drone images. The GCPs were used as means of tying the processed drone imagery to known coordinates and elevations and to bolster accuracy.

Table 3.1 Showing Ground Control Point Coordinate Data using WGS 84 UTM 21N Coordinate Projection System

Location Tag	Northing	Easting	Survey Elevation	Adjusted Elevation for MSL
1	766505.1	348843	18.42	2.79
2	766189.2	348491.5	16.86	1.23
3	766023.7	348632.2	19.03	3.40
4	766272	348737.5	17.59	1.96
5	766416.2	348716.1	16.52	0.89

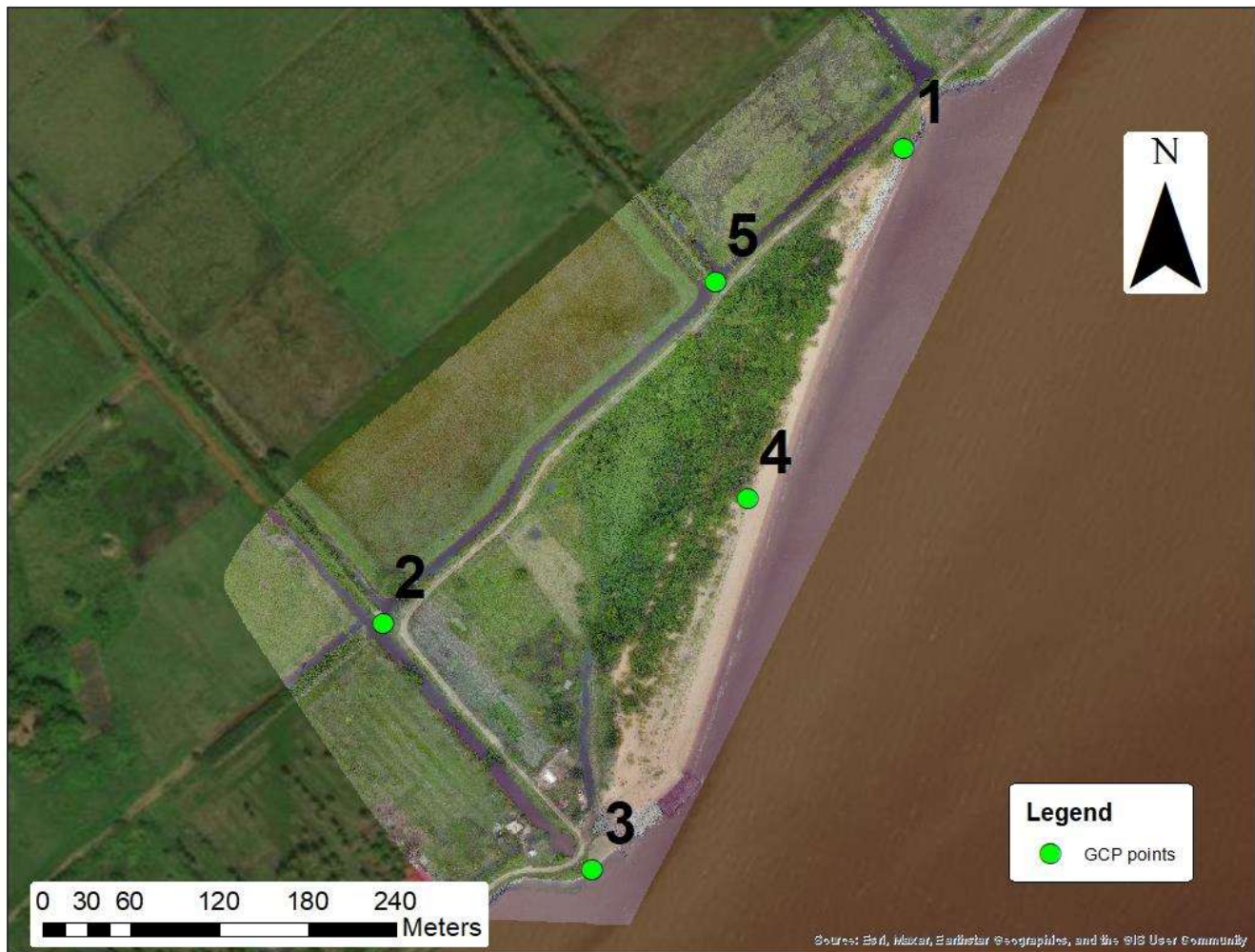


Figure 3.3: Location of GCPs on Leguan beach

The aerial survey revealed there is a berm to east of the project site. Its elevations ranged from 1.6 -3.7m above mean sea level. Behind the protection of the berm typical elevations within the project area and its surroundings ranged from approximately 0.4m to 2m above MSL. There were also instances of settling water observed that indicated small depressions and streams within the area that were below the MSL datum (See Figure 3-4)

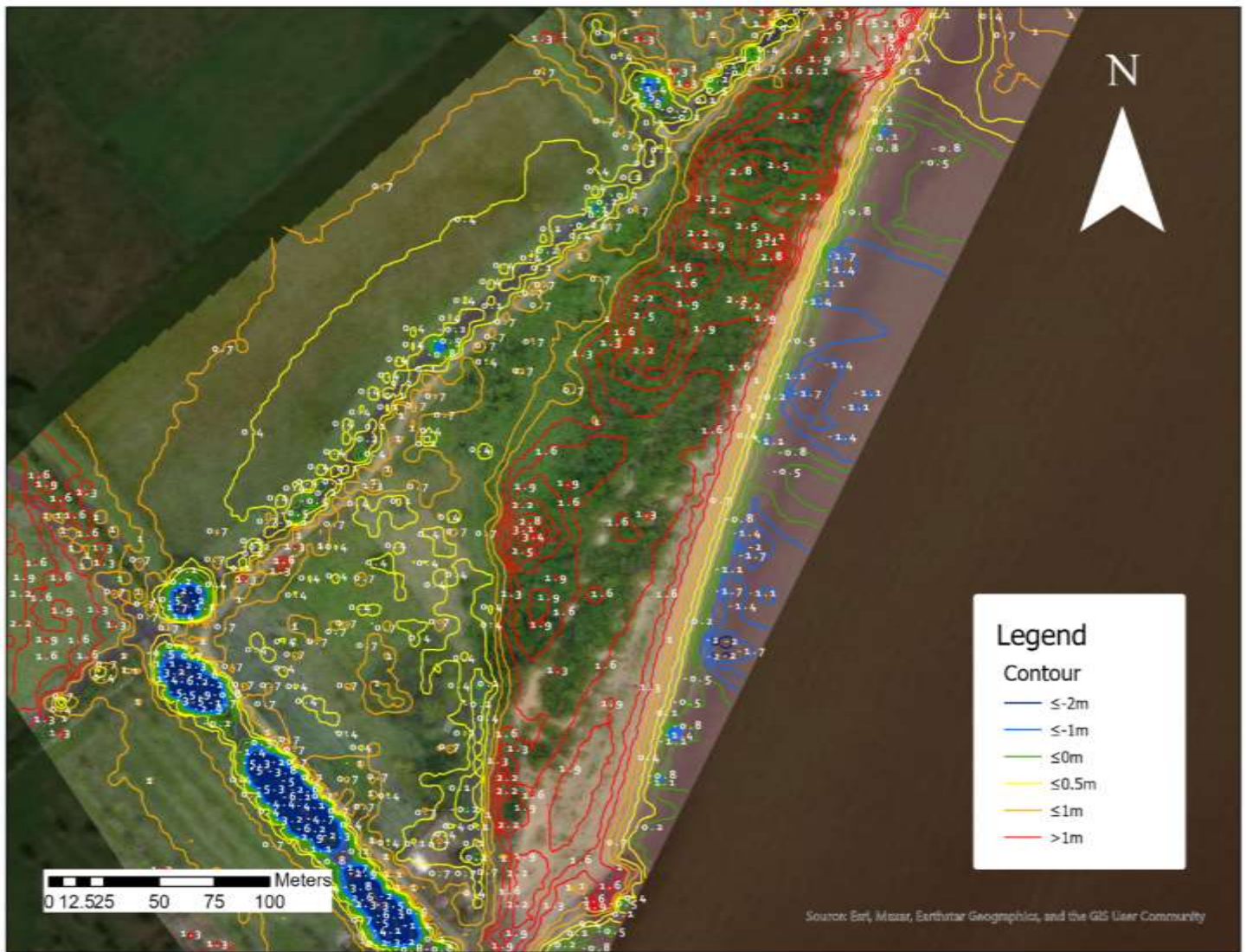


Figure 3.4 Contour Map generated from topographic elevations of Photogrammetry survey.

3.2.1 Bathymetry

3.2.1.1 Existing Chart and Databases

Bathymetric Data was collected from a myriad of sources for channel and ocean depths. For large and more expansive regions of the project model the main sources of depth included the following:

1. Merged Geotiffs from the General Bathymetric Chart of the Oceans (GEBCO)
2. Merged and referenced Navionics navigation charts for the Essequibo River.
3. Admiralty Charts from the UK Hydrographic Office.

Collected datasets were analysed and referenced to generate a final representative bathymetric profile of the river all tied to the same MSL datum.

3.2.1.2 CEAC Single Beam Sonar Survey (2022)

A bathymetric survey was performed by CEAC with a Single Beam Sonar to obtain more refined bathymetric data within a 2km proximity to the project area. This was to allow for more refined analysis of coastal and riverine processes within the nearshore environment of the site. The survey was performed on a series of gridded plan lines on which sonar depths were collected and processed (See Figure 3.5).



Figure 3.5 (Left) Garmin Echo Map used to log sounder outputs during the bathymetric survey, (Right) Profile and Plan lines used to collect bathymetric data.

The nearshore areas of the site are relatively flat with nearshore slopes of 2%, with depths averaging around 2m at 100 m offshore. The project area sits behind a berm at 3m elevation with the shoreline experiencing almost 3m of tidal range. Mangrove forests and sand banks were observed north of the project site indicating a very dynamic coastline. This bathymetric data is represented in Figure 3.6

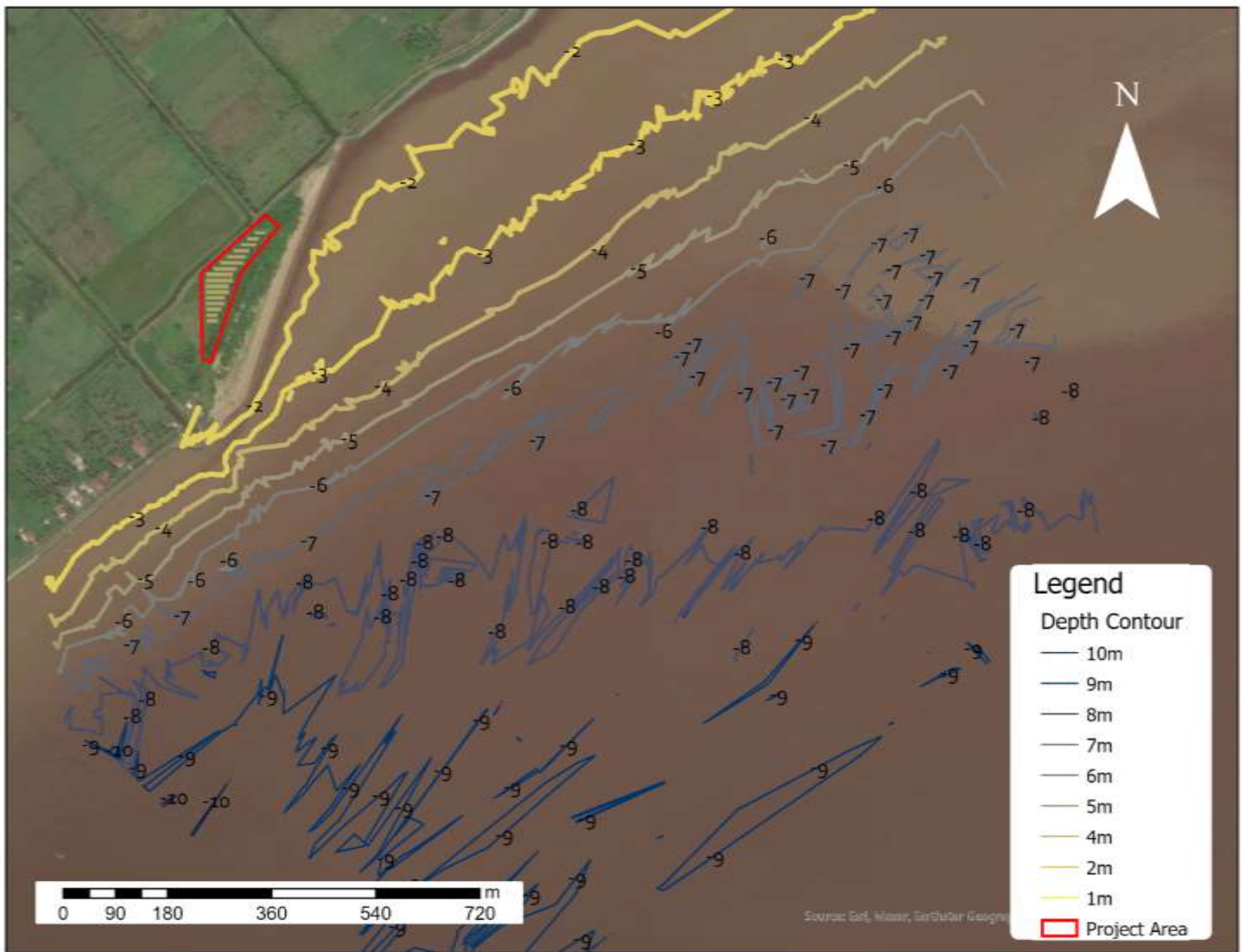


Figure 3.6 Bathymetric Map of the nearshore at the project area

3.3 Wind Data

Historical wind data were obtained from the George Town, Guyana station (SYEC) for the period January 1, 2016, to October 6, 2022. The data was analyzed, and wind roses were generated to determine the operational wind conditions. This data determined that north-easterly (NE) direction was the dominant wind direction with wind speed averaging 4.4m/s. Dominant speeds, however, range from 4.0– 9.9m/s.



Windrose Plot for [SYEC] Eugene F Correia

Obs Between: 15 Oct 2016 07:00 AM - 04 Oct 2022 08:00 AM America/Guyana

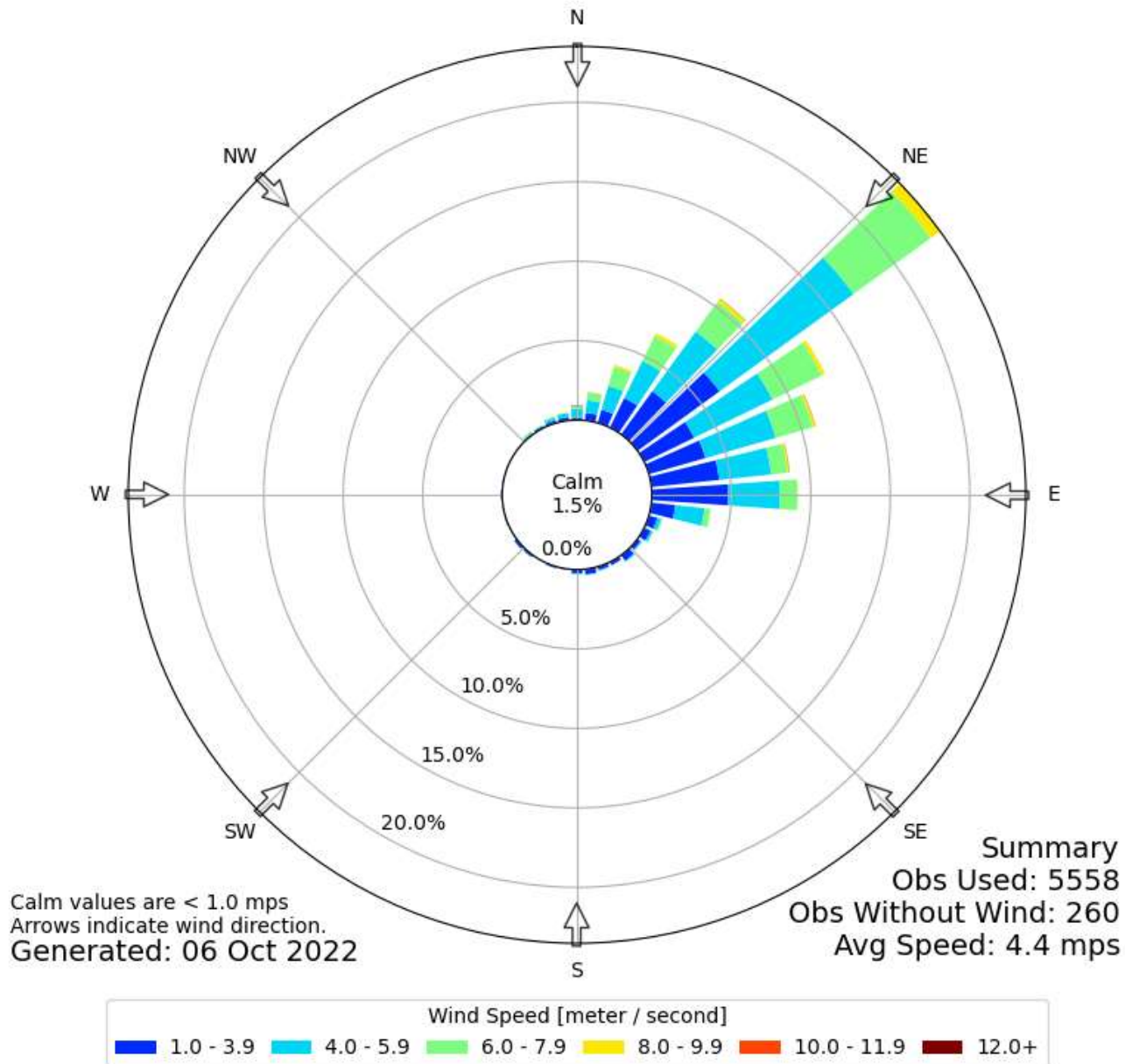


Figure 3.7 Wind Rose for the George Town for the period January 2016, to October 2022

3.4 Tides

Tides are the periodic rise and fall of sea levels caused by the gravitational forces exerted by the Moon and the Sun and the rotation of the Earth. It was important to correct the water depths for the bathymetric survey data by accounting for the tides during the survey and to inform the project water level calculations and its variations. The tidal range at the project location is approximately 3m during spring tides shown in Figure 4.2. The tidal range in this area is relatively large and was a major consideration in determining the flood risks to the site.

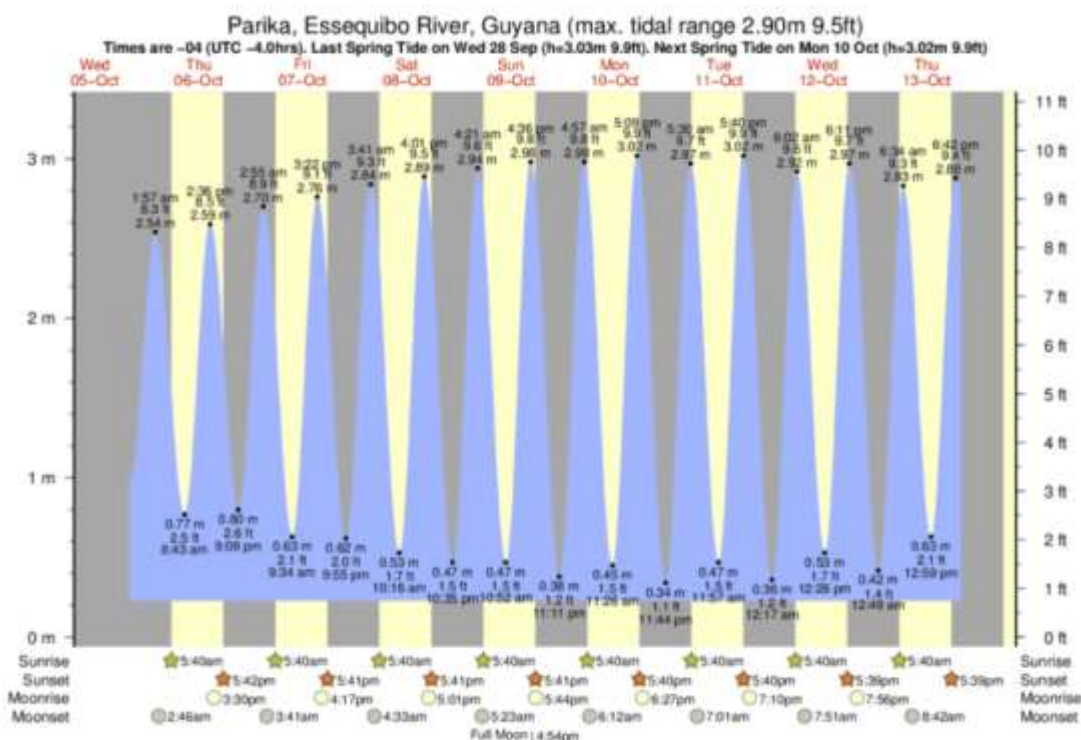


Figure 3.8 Tide Prediction (October 5 – 16 2022)

Table 3.2: Tidal Variations along the Coast of Guyana⁹

Tidal Summary		
	Spring Tide	Neap Tide
High water mark (HWM) (in relation to MSL)	+1.25	+0.58
Mean Sea Level (MSL)	0	0
Low water line (LWL)	-1.31	-0.61
Highest astronomical tide recorded	+1.58	
Future highest astronomical tide (with SLR)	+ 1.78	
Lowest astronomical tide recorded	-1.76	

⁹ U. Best et. al. (2022). Wave Attenuation Potential, Sediment Properties And Mangrove Growth Dynamics Data Over Guyana's Intertidal Mudflats: Assessing The Potential Of Mangrove Restoration Works. essd-14-2445-2022.pdf (copernicus.org)

3.5 Sediments Analysis

The existing beach grain size informs the long-term equilibrium beach profile. Sand samples were collected along the shoreline in October 2022 to determine the representative grain size. Six (6) samples were collected in the project area:

- i) at the berm
- ii) at the beach face
- iii) at one-meter depth

See Figure 2.6 below for the sand sample locations within the project. The grain size analysis uses the Unified Soil Classification System (USCS). The samples were dried and sieved using ASTM standard sieves, and the method of moments was applied to determine the mean grain size, skewness, and kurtosis. A tabular summary of the results is shown in Table 2.1. The grain sizes and their hydraulic properties are discussed in the subsections below.

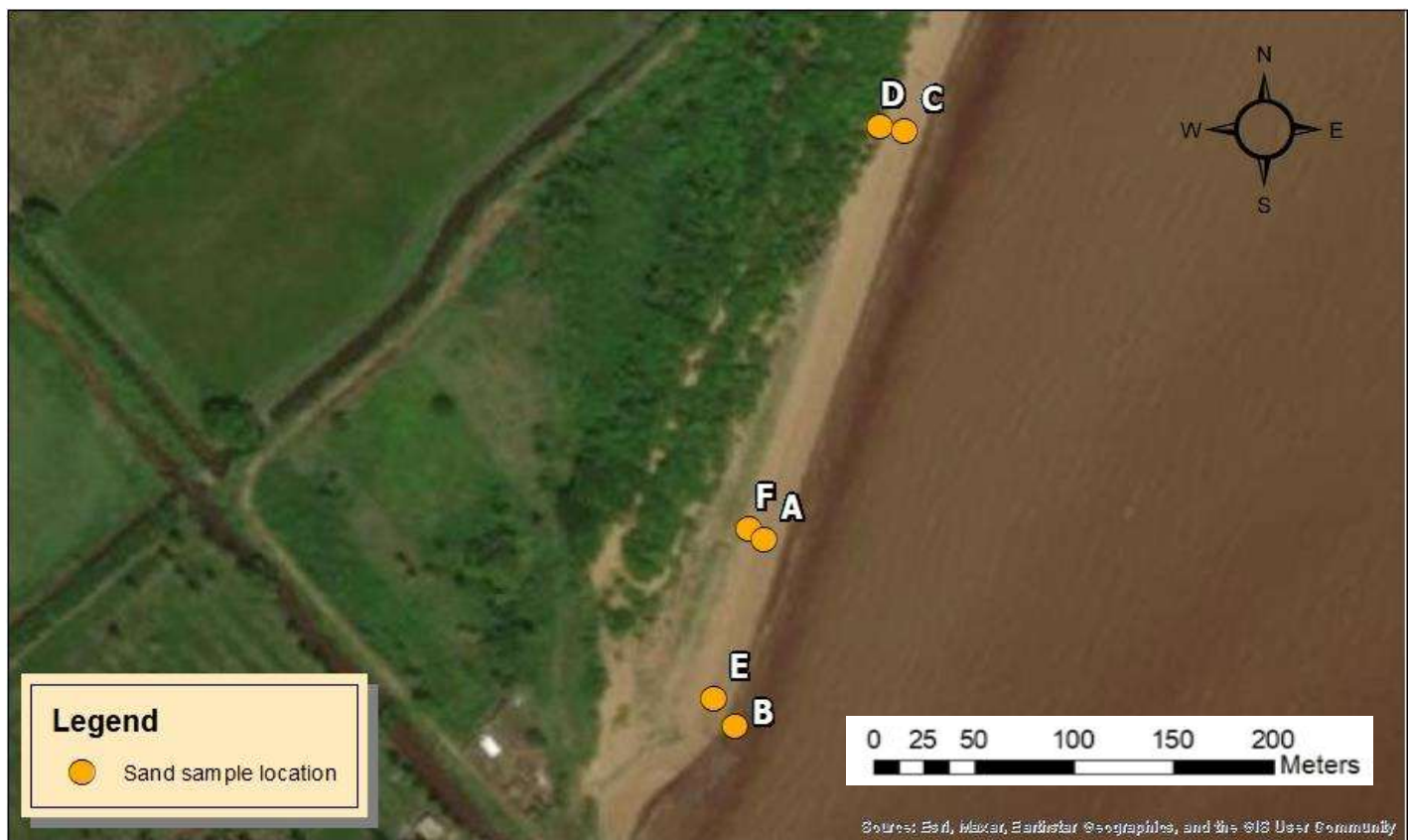


Figure 3.9 Sediment samples taken along the shoreline in the vicinity of the project site

The existing sand is predominantly coarse light brown/tan coarse sand. An external sieve analysis was conducted on the 6 samples and the sieve results can be seen in the appendix and are summarized in Table 2.1. The median sediment sizes range from 0.450mm (medium sand) to 0.885mm (coarse sand) with no silt content in any of the samples. The proposed beach nourishment will have to match this grain size or make further allowances to ensure the stability of the beach.

Table 3.3 Grain Size Analysis Results

Grain Size Analysis Results						
Sample ID	A	B	C	D	E	F
Location	Beach Face #1	1m Depth #1	Berm #2	Beach Face #2	Beach Face #3	Berm #1
Mean (mm)	0.710	0.885	0.525	0.450	0.842	0.521
Mean (phi)	0.493	0.176	0.929	1.152	0.248	0.940
Description	coarse sand	coarse sand	coarse sand	medium sand	coarse sand	coarse sand
Percentage silt	0.00%	0.01%	0.0%	0.0%	0.0%	0.0%
Percentage >0.06mm and <6.0 mm	99%	100%	100%	100%	100%	100%
Uniformity Coefficient	2.350	1.709	2.234	1.597	1.812	1.677
Standard Deviation	0.643	0.529	0.764	0.488	0.436	0.596
	moderately well sorted	moderately well sorted	moderately sorted	well sorted	well sorted	moderately well sorted
Skewness	0.952	0.507	1.224	2.902	0.879	1.249
	strongly positive skewed	strongly positive skewed	V. strongly positive skewed	V. strongly positive skewed	strongly positive skewed	V. strongly positive skewed
Kurtosis	0.798	1.413	0.927	1.301	1.046	0.854
	platykurtic	leptokurtic	mesokurtic	leptokurtic	mesokurtic	platykurtic
Median grain size (D50) mm	0.710	0.885	0.525	0.450	0.842	0.521

3.6 Hydrology

3.6.1 Data received and recovered

Data from a series of rain gauge stations across the South American countries of Brazil, Venezuela, Guyana, and Suriname were selected to perform a preliminary analysis of the climate and hence the distribution of precipitation in Guyana. Such a wide geographical span was utilized due to the large nature of the catchment affecting the outfall of the Essequibo River. The initial selection of gauging stations was then reduced from fourteen (14) to three (3) representative gauges. This process of elimination utilized a series of similarity, trend, and probability distribution analyses, that allowed the team to identify relationships or discrepancies between stations throughout various flooding events.

Upon completion the stations located at Georgetown, Mazaruni and Annai were selected and used as the precipitation input for the HEC-RAS floodplain model. The model applied the data spatially by converting it to a raster via the Inverse distance weighting (IDW) method.



Figure 3.10: Location of rainfall gauge stations in South America

Available rainfall data values varied for each station with the earliest year on record being 1840 (from the station in Georgetown) and the latest year being 2021 (from the La Bagatelle station on Leguan). From the collected dataset maximum

annual monthly precipitation values were tabulated for each station and used for similarity testing (between stations), trend testing and distribution fitting. See Sections 4.9.1.1 - 4.9.1.3 for details of the processes performed.

3.6.1.1 Similarity analysis: Station grouping and infilling

To consolidate the rainfall data and identify variances in precipitation due to distance, the two-sample Kolmogorov–Smirnov (KS) test (a non-parametric test measuring the goodness of fit) was conducted on the stations to determine whether the precipitation in two areas were similar in nature. After running the KS test, the precipitation recorded at the stations below were found to be similar in nature:

- i) Mazaruni and Annai (Southern)
- ii) La Bagatelle and Georgetown (Northern/Coastal)

With this knowledge, the rainfall data supplied by Georgetown was deemed to be somewhat representative of the precipitation that occurred on the island of Leguan (La Bagatelle) indicating that the regions experienced similar climate patterns over the span of the historical data. The Mazaruni and Annai station were also deemed to be similar, but Annai had recorded higher levels of precipitation in some instances. The cumulative distribution charts for similar stations are displayed in Figure 3.9. All other station data relationships when tested for similarity provided a negative. For example, Mazaruni vs Georgetown. This highlighted that there were clear variances in climate in regions of the catchment, especially between the northern and southern extents. From these findings, the catchment was split into three (3) precipitation regions represented by Mazaruni, Annai and Georgetown the primary sources for fluvial flood modelling.

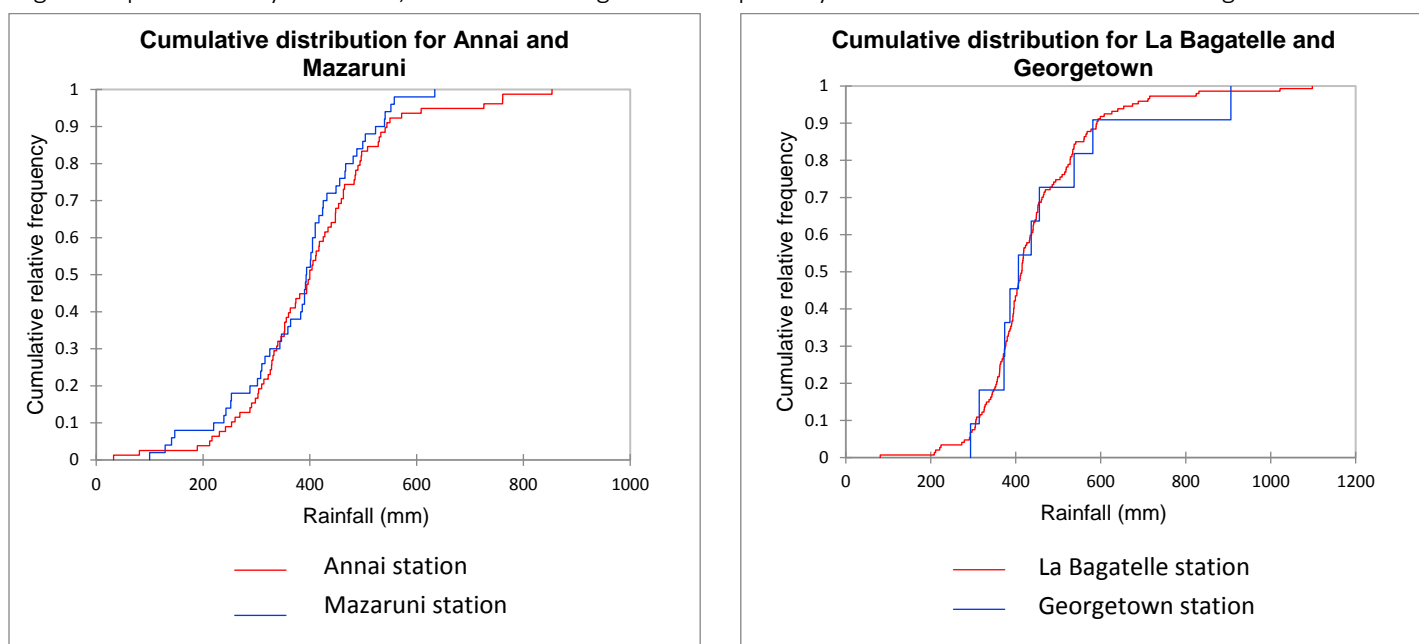


Figure 3.11: Cumulative distribution graphs from KS Test on Guyana rainfall stations

3.6.1.3 Stationarity analysis: Trends

The Mann-Kendall (MK) Test is another non-parametric test that analyses data over a period of time to determine if a trend can be found in the data series. A trend in the series may indicate climatic influences on the rainfall in that area. This test was conducted for the 3 primary stations where no such trend was found in the series. A sample graph from the Georgetown station is presented in Figure 3.11.

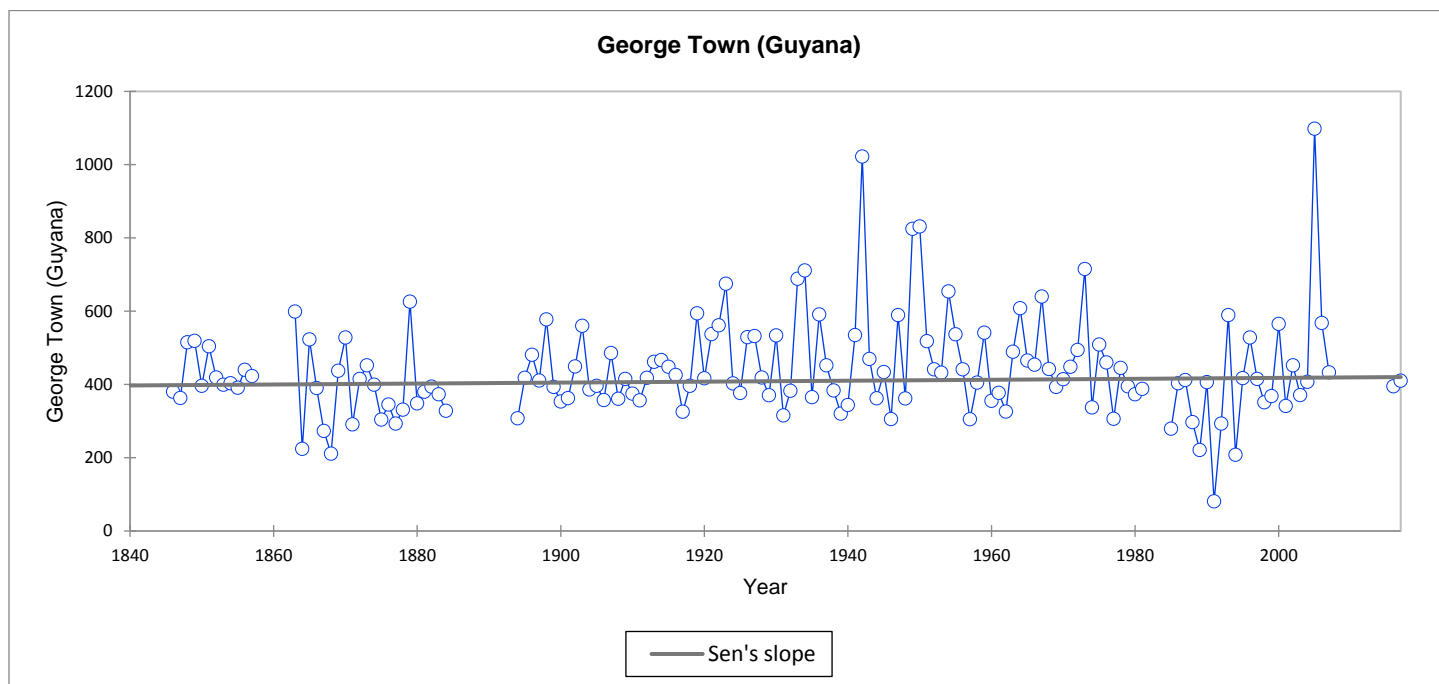


Figure 3.12: Sen's slope from the MK test for Georgetown

Further analysis was conducted to determine if there was any external climatic influence specifically from periodic fluctuations in the sea surface temperature. This would be attributed to the El Niño and La Niña southern oscillation (ENSO). The rain that fell during the El Niño and La Niña oscillation cycles were compared within each station data set using the KS test for the 3 primary rainfall data sets (Annai, Mazaruni, Georgetown). However, no such trend was found for any of the rainfall gauge data, allowing for justifiable use of the rainfall data.

3.6.1.4 Return period analysis: monthly and daily

Distribution fitting (DF) was also done for the 3 primary rainfall data sets (Annai, Mazaruni, Georgetown). This analysis forecasts the frequency of a certain distribution by fitting the individual characteristics of the data set to a known probability distribution. This allows for the most reliable predictions of the data set to be made. The method of moments was the technique used to fit the 3 primary data sets to their respective distributions. The Mazaruni rainfall data was fitted to the Logistic distribution (with a p-value of 0.901), Annai rainfall was fitted to Weibull distribution (with a p-value of 0.416) and Georgetown was fitted to the Fisher-Tippett distribution (with a p-value of 0.187). From these findings, the respective distribution prediction formulas were used to determine the expected rainfall for the 5-year, 10-year, 25-year, 50-year, 100-year and 250-year return periods. The formulas used are given below, where x is the probability of occurrence for any Return Period (RP) $\left(\frac{1}{RP}\right)$, and β and α are scale and location parameters respectively and μ is the distribution function. For each of the 3 rainfall stations, the respective precipitation values for each return period has been summarized in Table 3.5 and Figure 4.16

$$D(x) = \frac{1}{1 + e^{-(x-\mu)/\beta}}$$

Equation 1: Logistic distribution formula (used for Mazaruni)

$$D(x) = 1 - e^{-\left(\frac{x}{\beta}\right)^\alpha}$$

Equation 2: Weibull distribution formula (used for Annai)

$$D(x) = e^{-e^{-(\alpha-x)/\beta}}$$

Equation 3: Fisher-Tippett Gumbel distribution formula (used for Georgetown)

Table 3.4: Predicted rainfalls amounts for varying return periods based on distribution formulas

Return Period	Station Rainfall (mm)		
	Mazaruni Prizon	Annai	Georgetown
5	510	473	537
10	571	517	616
25	645	563	716
50	699	591	790
100	752	616	864
250	822	645	960

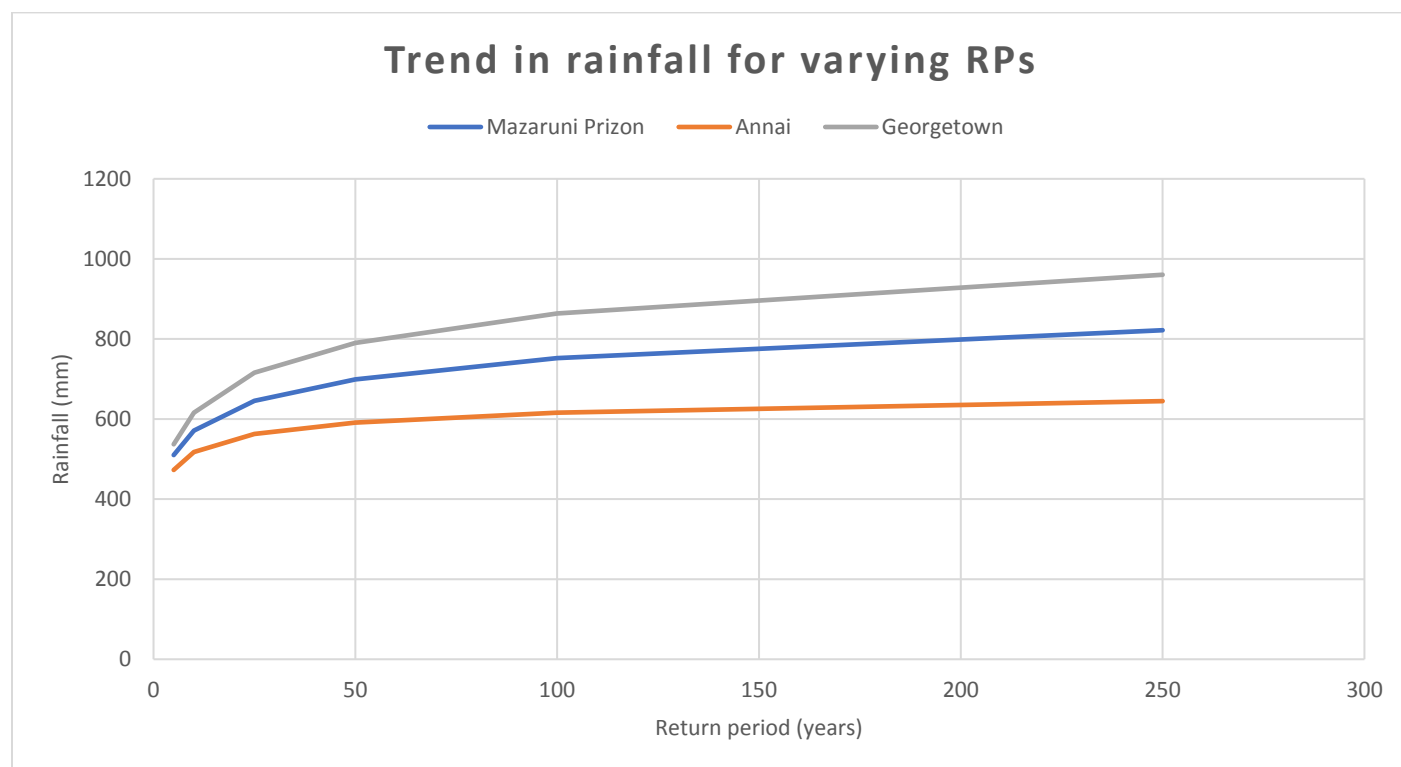


Figure 3.13: Trend in rainfall for the 3 primary stations for varying RPs

3.6.2 Monthly Rainfall mass curve

Establishing the expected monthly rainfall for each return period (RP) made it possible to revisit the station data and identify which year's monthly maximums corresponded to a similar amount of precipitation. From that monthly data, daily precipitation values were obtained to produce a mass curve and compare the distribution of the rainfall throughout that month. It was found that monthly precipitation values for lower RP events saw rainfall more uniformly across the month when compared to higher RP events which saw peaks of rainfall – most of that rain falling within only a few days. The La Bagatelle station (which had precipitation patterns similar in nature to the Georgetown station) was able to provide the necessary monthly data for this analysis. Its relative monthly distribution of rainfall was used as the standard for generating mass curves for the 3 primary stations. A sample graph from Georgetown showing the monthly rainfall distribution for its 5-year RP and 250-year RP events can be seen in Figure 3.12. Steady, lower amounts of rainfall can be clearly seen for the 5-year RP while a peak in rainfall can be seen in the 250-year RP. These rainfall values were then input into the HEC-RAS model to generate the computational mesh for the 2D flow area.

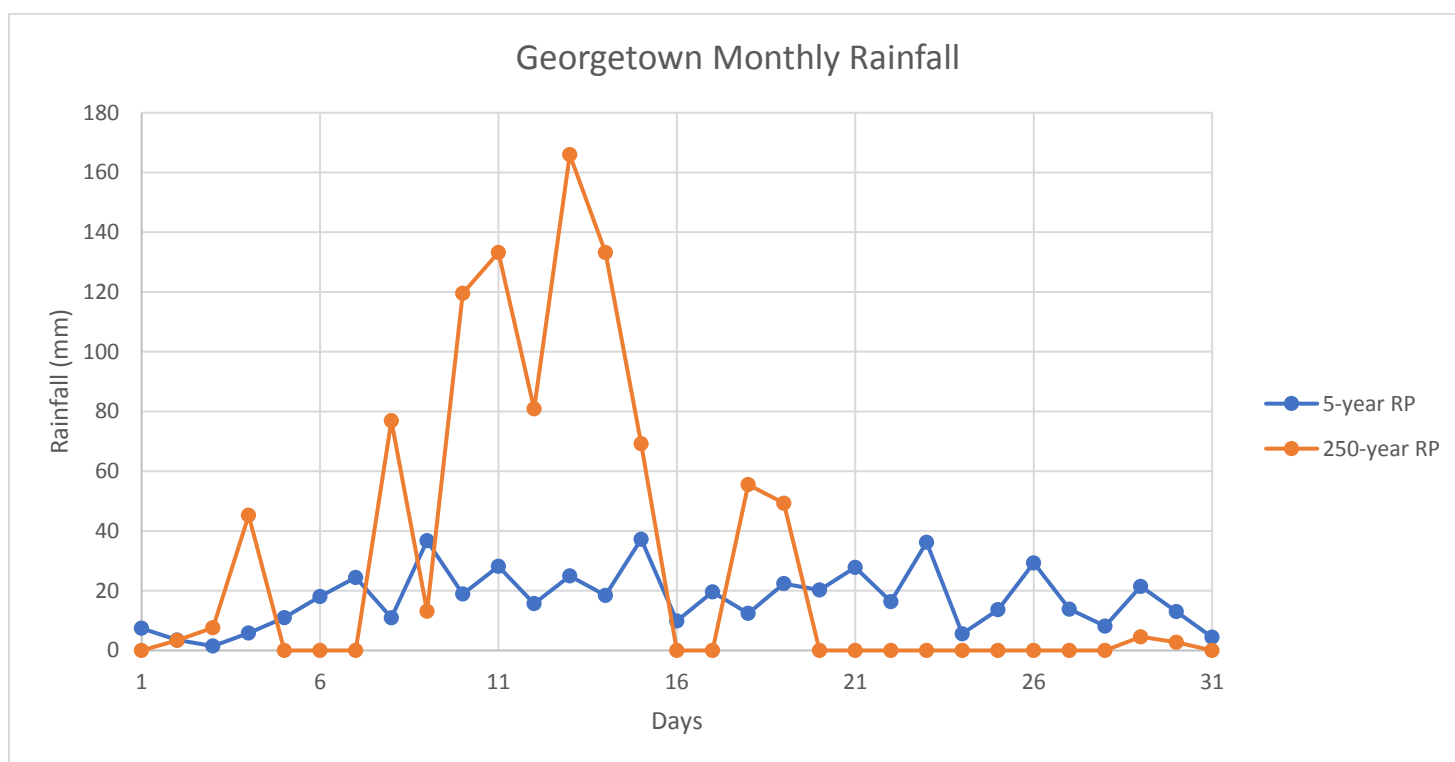


Figure 3.14: Georgetown monthly rainfall data for a 5-year and 250-year RP

3.6.4 Return periods for notable Guyanese flood events

Observing the rainfall data from the 3 primary stations, a closer look was taken at the rainfall flood events that devastated Guyana in 2005, 2015 and 2019, as discussed in Section 2.1 Flood Events and Damage History. Data availability remained a recurring issue when comparing the precipitation and devastation felt across Guyana.

The data extracted from the rainfall gauges provided the corresponding amount of rainfall for the aforementioned flood events and linked them to an RP based on the distribution fitting previously performed. This provides a general idea as to how frequently these disasters may occur. In 2005, the flood event commonly known as “The Flood” by locals, saw a monthly maximum of 1098mm of rain, indicating it to be in exceedance of 250RP occurrence.

Table 3.5: Expected RPs for notable flood events in Guyana

Rainfall Stations						
	Georgetown		Annai		Mazaruni Prison	
	Rainfall (mm)	RP (yrs.)	Rainfall (mm)	RP (yrs.)	Rainfall (mm)	RP (yrs.)
1942	1022	>250	424	2.85	530	6.21
1971	448	2.47	410	2.49	356	1.52
2005	1098	>250	N/A	-	490	4.07
2015	455	2.93	N/A	-	N/A	-
2019	373.9	1.66	N/A	-	N/A	-

3.8 Anecdotal

3.8.1 Data Collection and Results

To evaluate the accuracy of both floodplain modelling and coastal modelling process outputs from anecdotal field surveys were performed in October 2022. This allowed the engineer to determine the severity of hazards during a series of historical events associated with various return periods and compare them to the model outputs to validate and calibrate the results of the model.



Figure 3.15 Anecdotal Interview Location Map

Table 3.6 Summary of Anecdotal Interviews conducted October 19,2022 on Leguan Island

Anecdotal Form							
GIS ID	Interviewee Name	Age	Location (WP)	Time Living in Area	Extreme Weather Event	Inundation Depth (m)	Addition Comments
1	Ida	86	Amsterdam	70	Failure of Sea defence 2019, tide gate failure	<0.3m of flooding in the house	
2	Silvy Basdeo	96	Amsterdam	77	2019 tide gate failed during spring tide causing major flooding	0.2m to 0.25m of inundation in house	2019 tide gate failed during spring tide cause major flooding During high wind and spring tide high overtopping it raises the water level in the trench to the top of the trench
3	Haral	30	Amsterdam	30	Failure of Sea defense 2019, tide gate failure	0.05m of flooding in yard	No major flooding experience from either sea or the neighbouring canal
4	Haroon Waheed	65	Amsterdam	64	Failure of Sea defense 2019, tide gate failure	0.3m of flooding in house	Flood water recedes slowly during rainfall events due to the drains being blocked
5	Ray	60	Amsterdam	40	Failure of Sea defense 2019, tide gate failure	0.3m of flooding in surrounding properties	0.3m ft when sea defence fail in 2019
6	Harron	40	Amsterdam	10	Failure of Sea defense 2019, tide gate failure	0.05m of flooding in yard	If it falls 2 day consistently causes flooding from the trench majority of flooding in areas lower than road the school is on the level with the main road no flooding really affects the yard
7	Kevin	70	Amsterdam	50	Failure of Sea defense 2019, tide gate failure	0.6m of inundation in house	Rainfall neighbors flood they houses are filled they drains empty after 1 2 hours with the low tide
8	Joyce	72	Amsterdam	72	Failure of Sea defense 2019, tide gate failure		Flood this year when Koker failed and sea defence fail to get 1ft the trench needs to be cleaned
9	Patsy		Amsterdam		Failure of Sea defense 2019, tide gate failure	Mild flooding in yard	Only When koker and sea defence failed did no major flooding from rainfall

10	George	40	Amsterdam	40	Failure of Sea defense 2019, tide gate failure	Mild flooding in yard	
11	K Golding	70	Cemetery	70	Heavy Rainfall causes drains to overflow	Mild flooding on road	
12	Rodgers	80	Cemetery	60	Heavy Rainfall causes drains to overflow	Mild flooding in yard	
13	Rajnarine	65	Cemetery	65	Heavy Rainfall causes drains to overflow	<0.15m on rice field	Major flooding due to rain damaging the rice field due to an undersized clogged drain
14	Nandlall		Cemetery	50	Heavy Rainfall causes drains to overflow	>0.1m on road	Kocher not working effectively causing backflow when the rainfall is heavy last flooded last week's complaints drain. Blocked and undersized
15	Wickham	48	Cemetery	47	Heavy Rainfall causes drains to overflow	0.15m of flooding in house	Henry drain need to clean rain fall water enters his home with about 6 inches of water
16	Dennis	60	Stelling	60	Mild overtopping in high spring tide and wind scenarios	0.1m of flooding on roads	No major flooding due the rains waves Overtop during high tide to winds causes overtopping flooding the areas close to sea defence
17	Ashley		Stelling	35	Mild overtopping in high spring tide and wind scenarios	0.3m of flooding in yard	
18	Henry		Success	51	Heavy Rainfall causes drains to overflow	0.15m of flooding in house	
19	Baba		Success	65	Heavy Rainfall causes drains to overflow	0.1m of flooding in house	

3.8.2 Model Comparison

The majority of accounts provided in the anecdotal interviews gave significant insight into precipitation events that occurred during 2019. Through comparison of raster outputs with resident accounts, the model was deemed to be within a typical range of 13-50% of the values recorded within the same area. Through the application of the excel correlation function below:

$$\text{Correl}(X,Y) = \frac{\sum (x - \bar{x})(y - \bar{y})}{\sqrt{\sum (x - \bar{x})^2 \sum (y - \bar{y})^2}}$$

Figure 3.16 Showing the correlation function used to derive model output and anecdotal inundations

the outputs were deemed to have a correlation of 0.8 which lies within the range of a good to a very strong relationship. A general observation of the outputs also revealed that the inundation depths typically ranged from 0.1-0.6m in both data sets. As such the model outputs were deemed acceptable.

Table 3.7 Comparing observed and modelled flood depth for 373mm event

Interview ID	Observed Flood Depth (m)	Modelled Depth (m)	Variation
1	0.3	0.316	5%
2	0.25	0.286	14%
4	0.3	0.45	50%
5	0.3	NA	
6	0.05	0.35	600%
7	0.6	0.615	3%
8		0.62	
14	0.1	0.16	60%
15	0.15	0.13	-13%
Correlation		0.8018	

Table 3.8 Showing relation of correlation function to qualitative description

Correlation	
Negligible	0.1
Good or Moderate Correlation	0.65
Very Strong Relationship	0.9

The general coarseness of the data used for terrain outside of the project area meant that aligning the locations with the exact points of flooding was unlikely. The general accuracy of point comparisons was done with a 30m radius of accuracy which aligns the terrain model cell size.

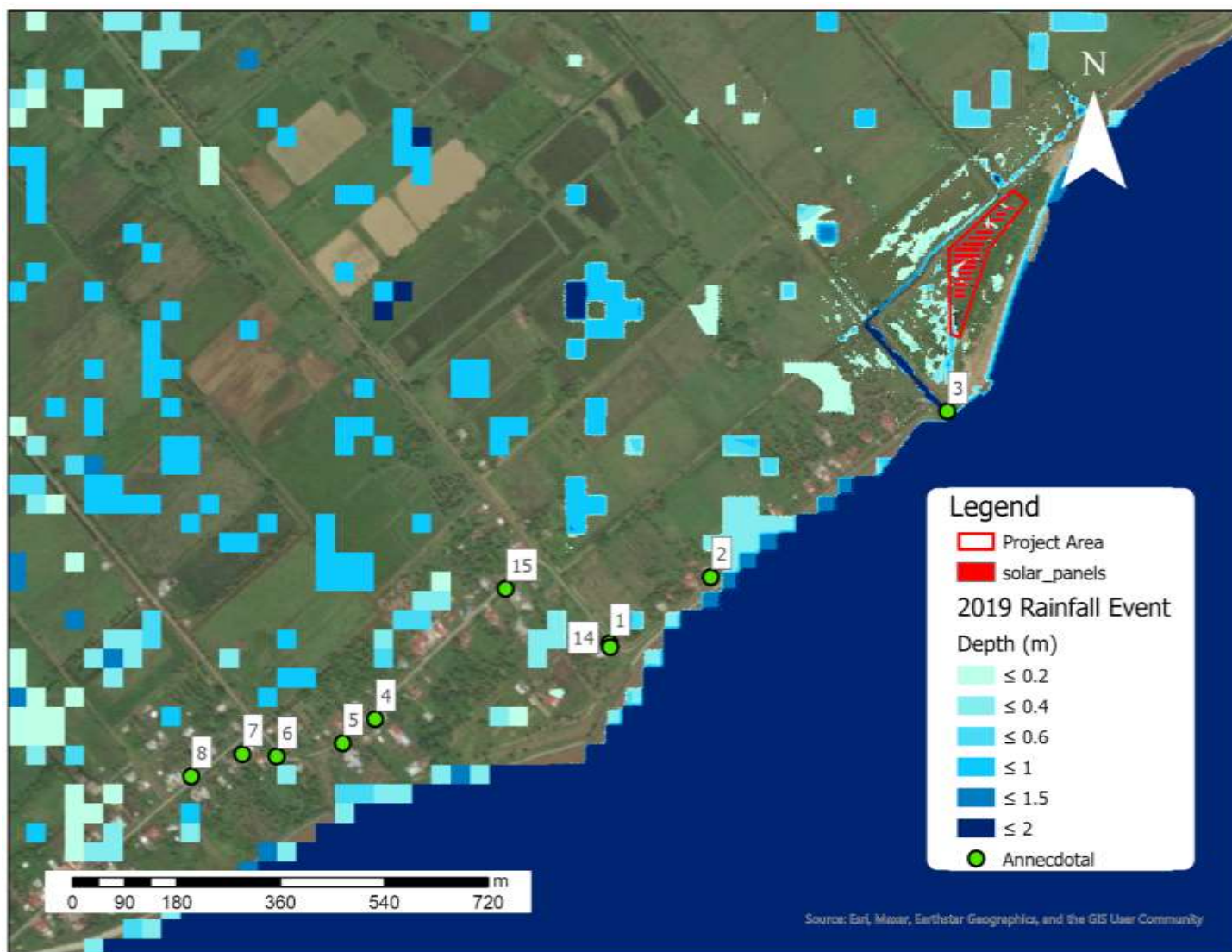


Figure 3.17 Map Comparing Modelled flood depths vs observed flood depths.

3.9 Summary

Topographic data suggested that site elevations ranged from as low as 0.4m up to 2m, which included several low-lying zones susceptible to ponding. In from of the site was protective berm area that consisted of elevations between 1.6-3.4m. Beyond this there was a nearshore area with an overall bathymetric slope of 2% down to a 9m depth. Offshore depth at the opening of the Essequibo raised to 1m and provide a sheltered wave environment from deep-water waves, but the possibilities of surge and longshore currents. The tide range is relatively large at 3.1m and will be increased with SLR.

The sediments in the profiles are generally coarse and conform to the (Equilibrium Beach Profile) EQBP. The profile is generally stable. Hydrological and statistical analysis suggest that significant precipitation events range from 470mm to 980mm for 5-year RP to 250-year RP events. Precipitation events were observed to be more severe in coastal areas and were typically associated with high damage events. To test model predictions a calibration was done will similar precipitation as experienced in 2019. The anecdotal findings for 2019, strongly supported the model predictions when

compared. Both suggested wide spreads “ponding” from the drainage systems. With typical inundation depths from 0.1-0.6m.

4 Hazard Assessment

4.1 Hydrology and Floodplain Modelling

4.1.1 Model Input

The HEC-RAS model was used to simulate the hydrological response of the contributing watershed to various month-long rainfall events. HECRAS uses a 2-D flow area model to transform precipitation inputs into flows influenced by topographic and soil data parameters. The generated flows indicate the fluvial flows and level of inundation experienced over the modeled flood plain.

Key features (inputs) used to do the assessment are as follows:

1. JAXA Topographic Data Topographic data (30m Geotiff).
2. GEBCO Bathymetric Data of River and Ocean.
3. CEAC Topographic and Bathymetric Survey Data
4. Historical and Projected Extreme Rainfall (Return Periods 5Yr-100Yr)
 - a. Climate Change Knowledge Portal – World Bank Group
 - i. Largest 1-Day Precipitation.
 - ii. Largest 5-Day Precipitation.
 - iii. Largest Monthly Cumulative Precipitation.
 - b. Daily precipitation - La Bagatelle Leguan (2011-2021)
 - c. Daily precipitation - Mazaruni Prison (1986-2014)
 - d. Monthly Precipitation (3 Gauge Stations, Georgetown, Mazaruni, Annai)
5. Flow Hydrographs (Min, Avg Max)
 - a. Water Resource Assessment of Guyana – US Army Corps of Engineers.
 - i. Plantain Island Station – (Essequibo River)
 - ii. Kamaria Falls Station - (Cuyuni River)
 - iii. Apaikwa and Hillfoot Station (Mazaruni River)
6. Land Cover and Soil Type
7. Tidal Variation from NOAA tidal charts.

For this assessment, the 5-Year, 10-Year, 20-Year, 50-Year and 100-Year present and future events were modelled to determine the impact extent of fluvial flooding on the Island of Leguan and more specifically the project area.

4.1.1.1 Climate Change Analysis

Future climate projections are based on representative concentration pathways (RCPs). RCPs are factor amalgamated greenhouse gas emission (GHG) scenarios used by the Intergovernmental Panel on Climate Change (IPCC), which categorizes possible future climates of the world. A few factors weighed into the scenarios include energy use, economic activity, and land use (Intergovernmental Panel on Climate Change 2018). There are four (4) defined scenarios, namely RCP2.6, 4.5, 6, and 8.5. Each scenario represents a future subjected to a specific radiative forcing value. The IPCC provides a probability ratio of heavy precipitation (historical vs future) as a function of global warming and event probability (Figure 3), which was utilized to determine climate change factors for the 2-yr, 25-yr, 50-yr, 100yr and 250yr RP. Climate change factors (CCF) were applied to the present climate to determine extremes for peak day, 5-day and cumulative monthly rainfall depths.

The Climate Change Knowledge Portal for Development Practitioners and Policy makers developed a return level plot offering a link between frequency and event magnitude, which was compounded with the IPCC frequency probability ratio to predict the future extreme.

For the purposes of this project, Climate change projections were done as high as RCP8.5 scenario up to mid-century (2041-2070). RCP 8.5 assumes high GHG emissions with low global behavioral change towards GHG mitigation. This scenario is denoted by a GHG range > 1000 parts per million (ppm).

4.1.1.2 Rainfall Data

Depth of rainfall for various return periods were provided by various sources. These include monthly average rainfall hydrographs from the Hydrometeorological Service of Guyana, Historical and Future Projections for Extremes rainfall events (1-Day Peak, 5 -Day Peak and Cumulative Monthly Peak) from the Climate Change Knowledge Portal generated by the World Bank Group and Daily and Monthly Precipitation Depths from individual rainfall gauges as discussed and analyzed in Section 4.9. Via application of 2° global temperature rise targets akin to the RCP2.6-RCP4.5 scenarios, precipitation values determined prior were then scaled to account for increased severities that may be expected due to the impacts of climate change. See Figure 5.1 and Table 5.1.

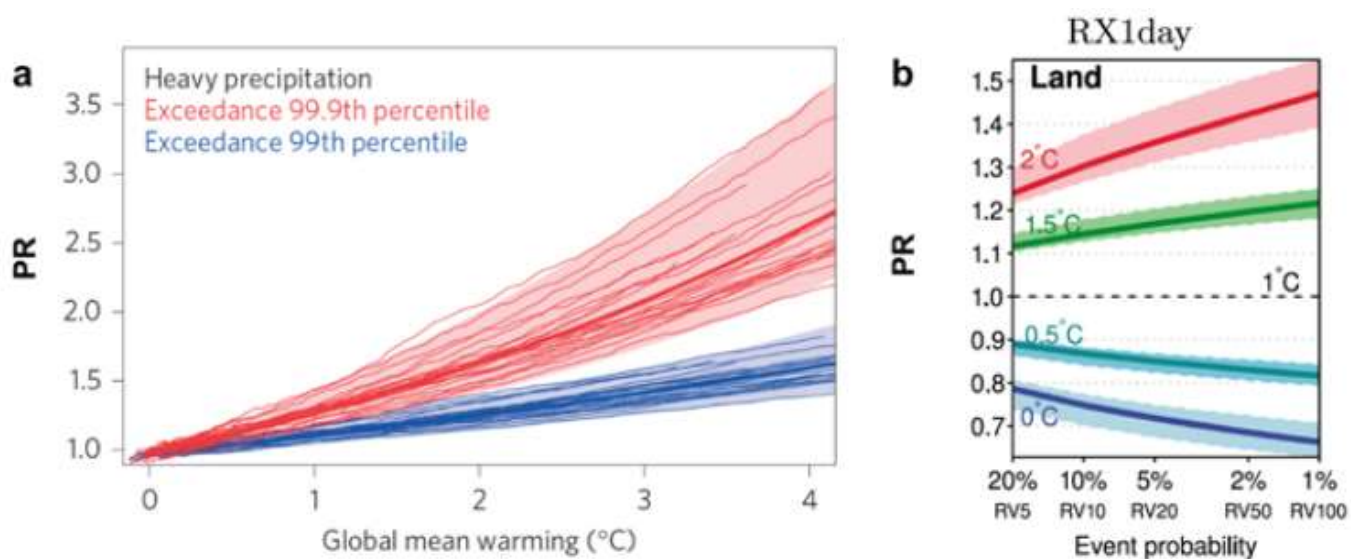


Figure 3.10 | Probability ratio (PR) of exceeding (heavy precipitation) thresholds. (a) PR of exceeding the 99th (blue) and 99.9th (red) percentile of pre-industrial daily precipitation at a given warming level, averaged across land (from Fischer and Knutti, 2015). (b) PR for precipitation extremes (RX1day) for different event probabilities (with RV indicating return values) in the current climate (1°C of global warming). Shading shows the interquartile (25–75%) range (from Kharin et al., 2018).

Figure 4.1 Probability ratio of heavy precipitation as a function of global warming and event probability (IPCC 2018).

Table 4.1 Showing station gauge precipitation adjusted for 2-degree Celsius temperature increase scenario.

RP	Mazaruni		Annai		Georgetown		Average Percentage Change
	Present	Future	Present	Future	Future	Future	
5	510.12	533.30	472.83	494.34	537.00	565.61	5%
10	571.32	591.01	517.41	526.00	616.05	644.15	3%
25	645.34	667.28	562.91	567.85	715.93	747.97	3%
50	699.20	724.99	591.22	599.51	790.03	826.51	3%
100	752.28	782.69	615.99	631.16	863.58	905.05	4%
250	821.88	858.97	644.76	673.01	960.42	1008.88	5%

For the purpose of flood modelling, it was necessary to provide a representative distribution of precipitation within the month on individual days so as to mimic the events that actually caused flooding. A review of the precipitation patterns which occurred during the 2005 flood event at the Georgetown station, indicated that within the peak month, there was a period of approximately one week wherein a majority of rainfall had been concentrated. As such a precipitation mass distribution was modelled using a unimodal curve with a peak at its centre. During that month, the Georgetown gauge station recorded precipitation of 1098mm, which was in excess of the 250RP.

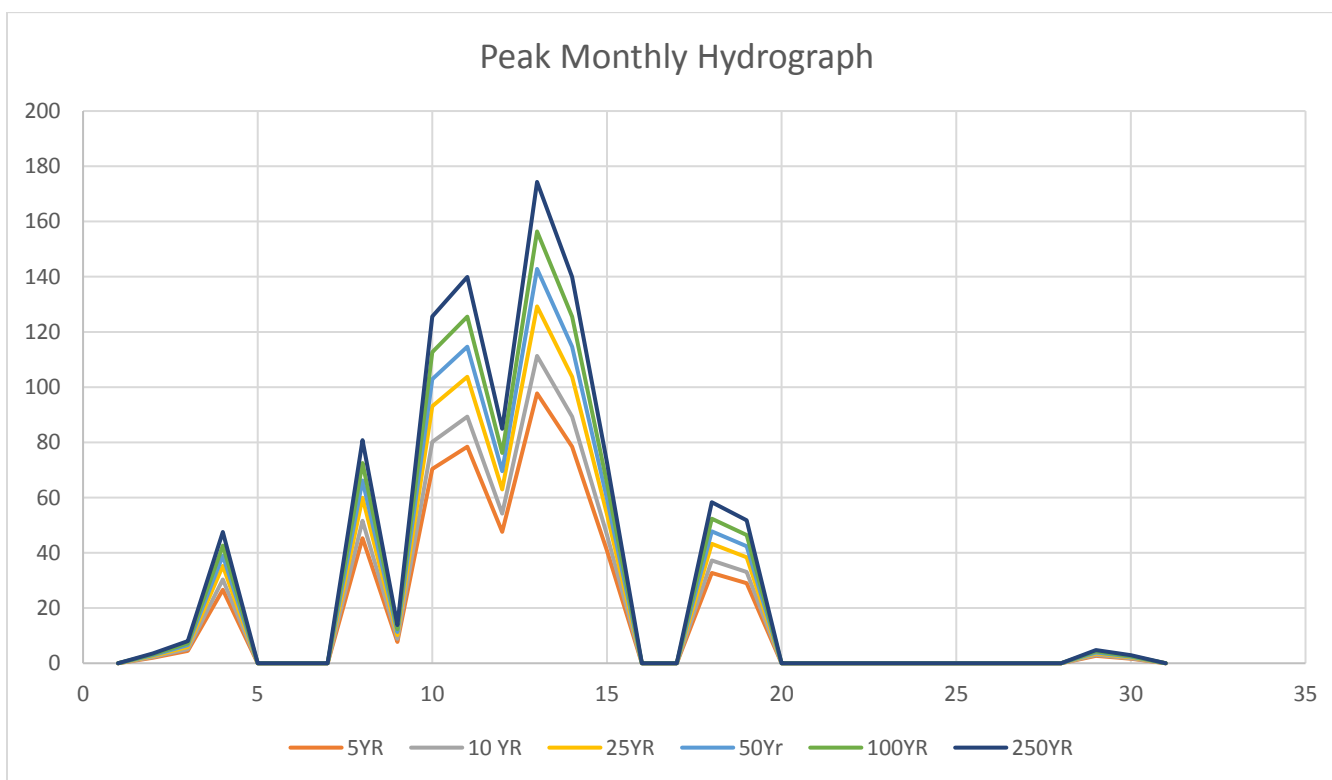


Figure 4.2 Monthly mass curve hydrograph for the future climate return periods ranging from 5 -250 years at the Georgetown gauging station

Checks at the Mazaruni and Annai stations during the 2005 event and a similar rainfall occurrence in 1942 indicated that the stations averaged 510mm and 424mm respectively. These represented 3-5Yr RP events. This observation was used as the premise for precipitation variation in the model. Wherein the modelled scenarios saw the Mazaruni and Annai stations being maintained at or below 5Yr RP events, while the Georgetown stations varied from 5RP -250RP events.

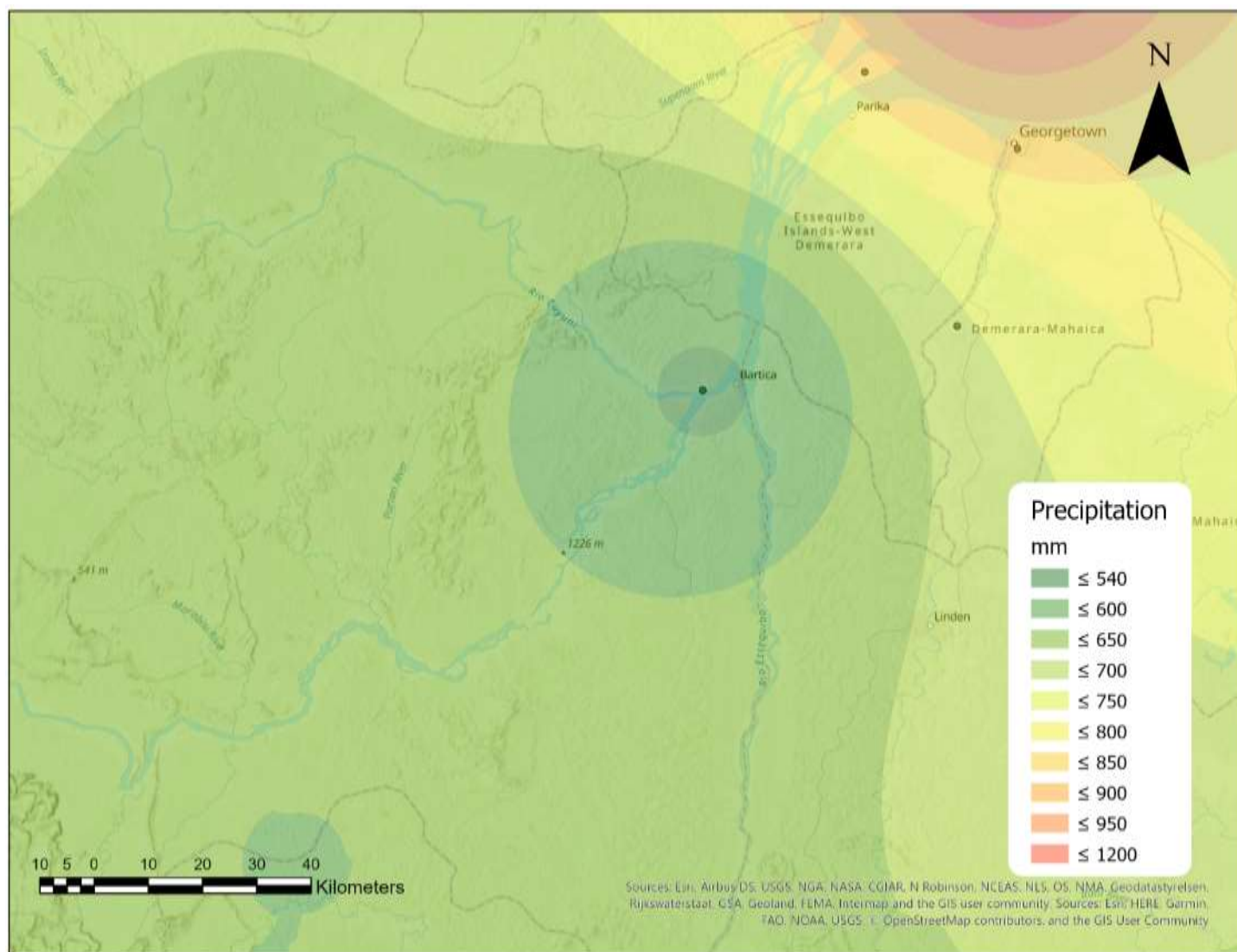


Figure 4.3: Cumulative monthly precipitation for 250-year RP rainfall event

4.1.1.3 Topography and Bathymetry

The merged topographic and bathymetric data was utilized firstly to used delineate used to delineate and establish the boundaries of catchments contributing to the Essequibo River outlet. See Figure 5.4 This of course provided an estimation for the surface area within Guyana that possibly contribute to flows at the Essequibo, which set the basis for the size of the 2-Dimensional analysis to mesh size. The area of the 2D mesh was determined to be 149150 km². The impact of varying reaches on the Essequibo will of course occur via significantly different timelines though due to the large span of the area. The terrain within itself was also utilized in the mapping and conveyance analysis of flows when projected into the HEC-RAS computational model.

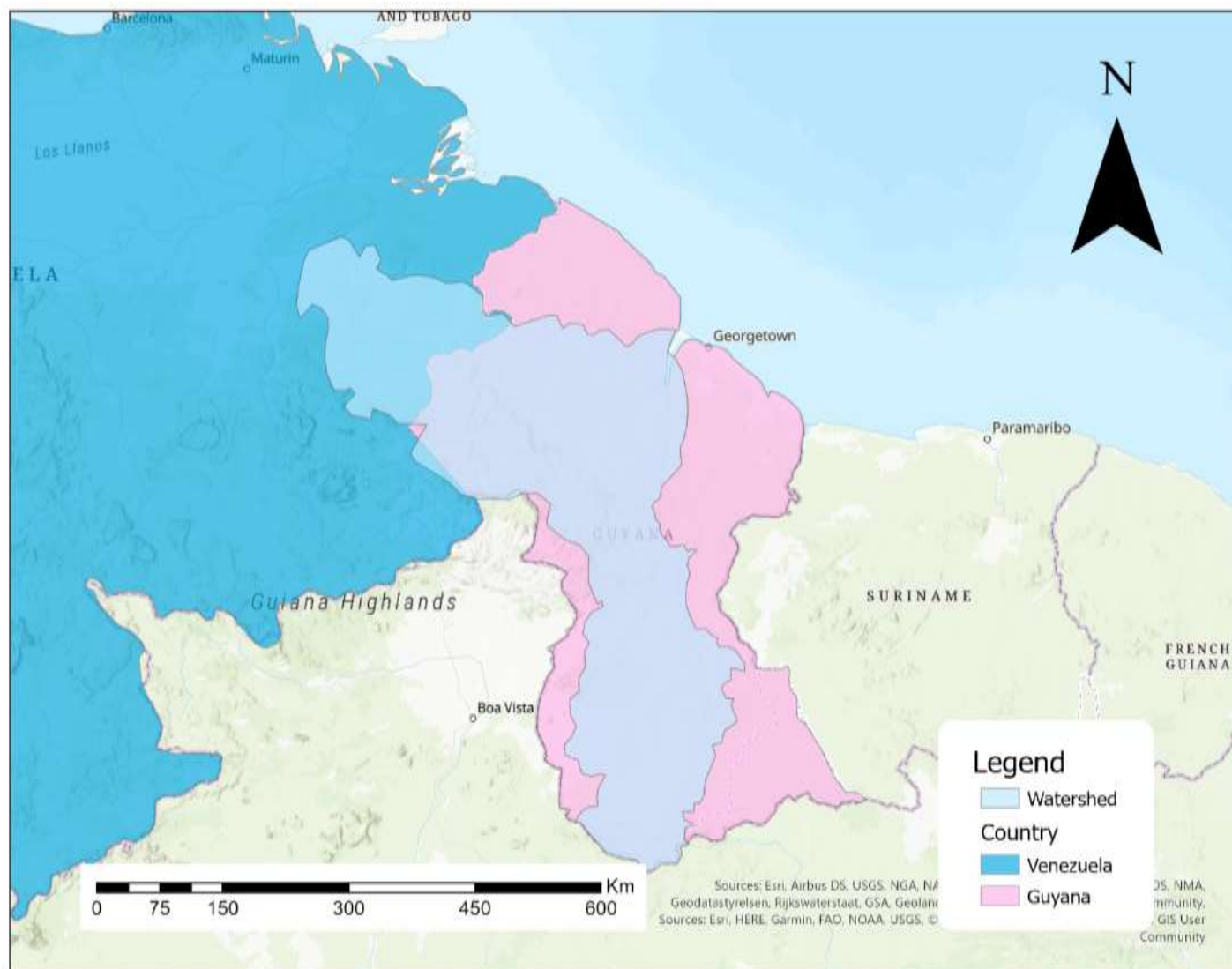


Figure 4.4: Essequibo River Contributing Watersheds

4.1.1.4 River Flow Hydrograph.

Flow hydrographs mimicking the average river flow conditions were used to initialize normal riverine flows. This was done by applying flows to the contributing reaches leading into the Essequibo at gauge stations identified from the Water Resource Assessment of Guyana (1998) done by the US Army Corp of Engineers. The selected gauges include the Plantain Island station located in the Essequibo River, the Kamari Falls station from the Cuyuni River and the Hillfoot station of the Mazaruni River.

Table 4.2 Showing extract of hydrograph flow data from Water Resource Assessment of Guyana

River Name	Gaging Station	Maximum Daily Flow(CMS)	Mean Daily Flow (Annual Average)
Essequibo	Plantain Island	8013	2224
Cuyuni	Kamaria Falls	5394	1062
Mazaruni	Hillfoot	2609	1145

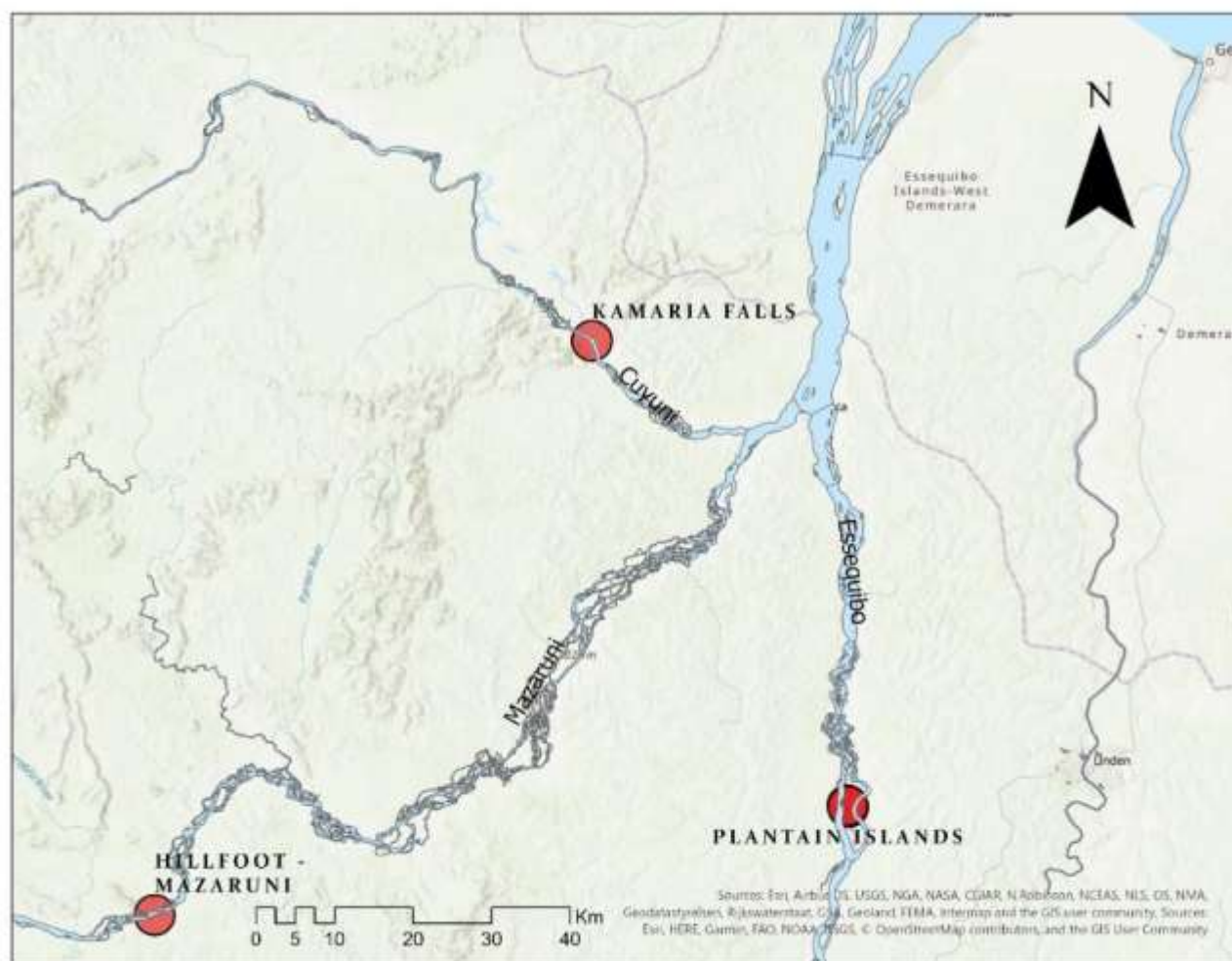


Figure 4.5 River Flow Gaging stations identified in Guyana Water Resource Assessment contributing the Essequibo River

4.1.1.5 Land Use

An overview of the landscape in Guyana indicates a heavily forested region with thick dense areas of medium to tall evergreen forests. Satellite Imagery, used to identify built-up areas showed that majority of housing was skewed to the northern coasts of the country. Forested regions accounted for upwards of 83% of the land cover. Further image classification indicated that 8% was covered by open and scrubbed savannahs, 3% by swamp forest, 2% by cropland and open water respectively and less than 1% by built up areas. The remainder is expected to consist of scrubland, barren land and other rural infrastructure networks not easily visible. Through weighting the manning's coefficients of each area, a value of 0.09 was determined, however the modelled Manning's selected was 0.06. This was to factor in the influence of major rivers significant conveyance capacity. Typically, major rivers possess a Manning's coefficient of 0.035.

Table 4.3 Preliminary Land use breakdown derived from image classification

Land Cover Category	Area (ha)	Manning's Coefficient	Percentage Impervious (%)
Built/Buildings and other infrastructures	300179.28	0.011	90
Mangrove/Swampy Forest	600600.87	0.100	50
Cropland/Plantation (Cultivation)	498628.94	0.050	10
Med/Tall Evergreen Forest	17422000.01	0.11	0
Barren/Dry Area	-	0.020	0
Savannah/Grassland/Shrubland	1695667.95	0.05	0
Open Water	435550.77	-	-

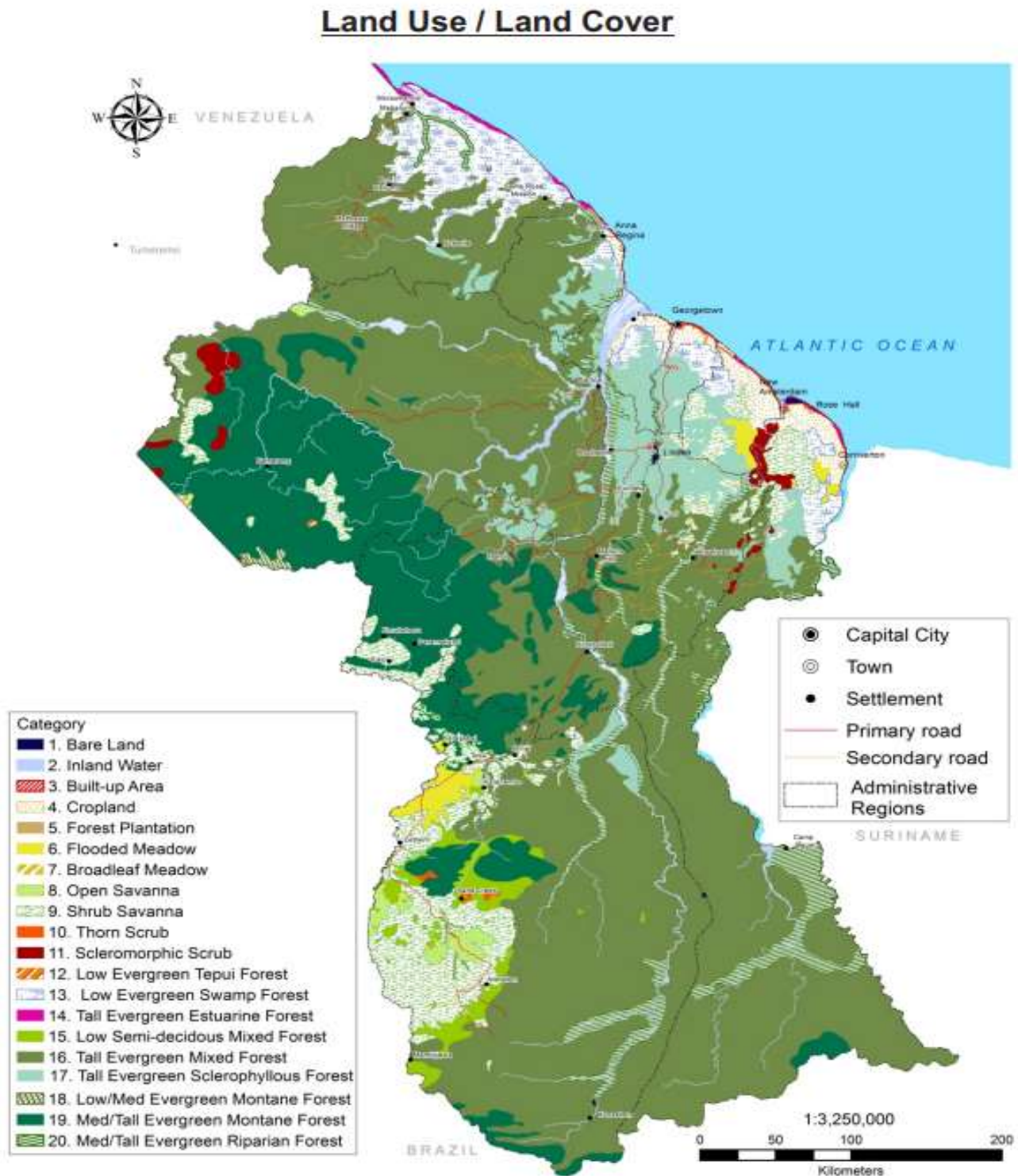


Figure 4.6 Land Use Map prepared by the ministry of natural resources and the Guyana lands and surveys commission.

4.1.2 Flood Plain Model Results

4.1.3 Site-Specific Flooding

A review of the Leguan Solar PV Project Report provided the approximate location of the solar farm site and the placement of major infrastructure. One key feature near the site was a koker. Kokers are common infrastructure throughout the island of Leguan and coastal regions of Guyana due to the typically low-lying nature of the area. The kokers are integrated into the drainage system to prevent elevated water levels from high tide events from flowing landward. As such the team found it key to investigate 2 scenarios, one where the koker was left open (possibly due to a failure) and one where the koker had to remain closed due to high tide.

Inundation plots from these scenarios modelled in HEC-RAS were overlain to provide a relative site impact. Where the gates were closed during a high tide event, inundation depth ranged from 0.3m in the 5-year rainfall event to 0.8m in the 250-year rainfall event. These depths were determined using the project area low point of 0.4m above MSL in relation to observed water surface elevations that propagated inland. It was also observed that flood extent increased from 12% of the project area during the 5 RP event to 59% during the 250 RP event. See Figure 4.6 - 4.7

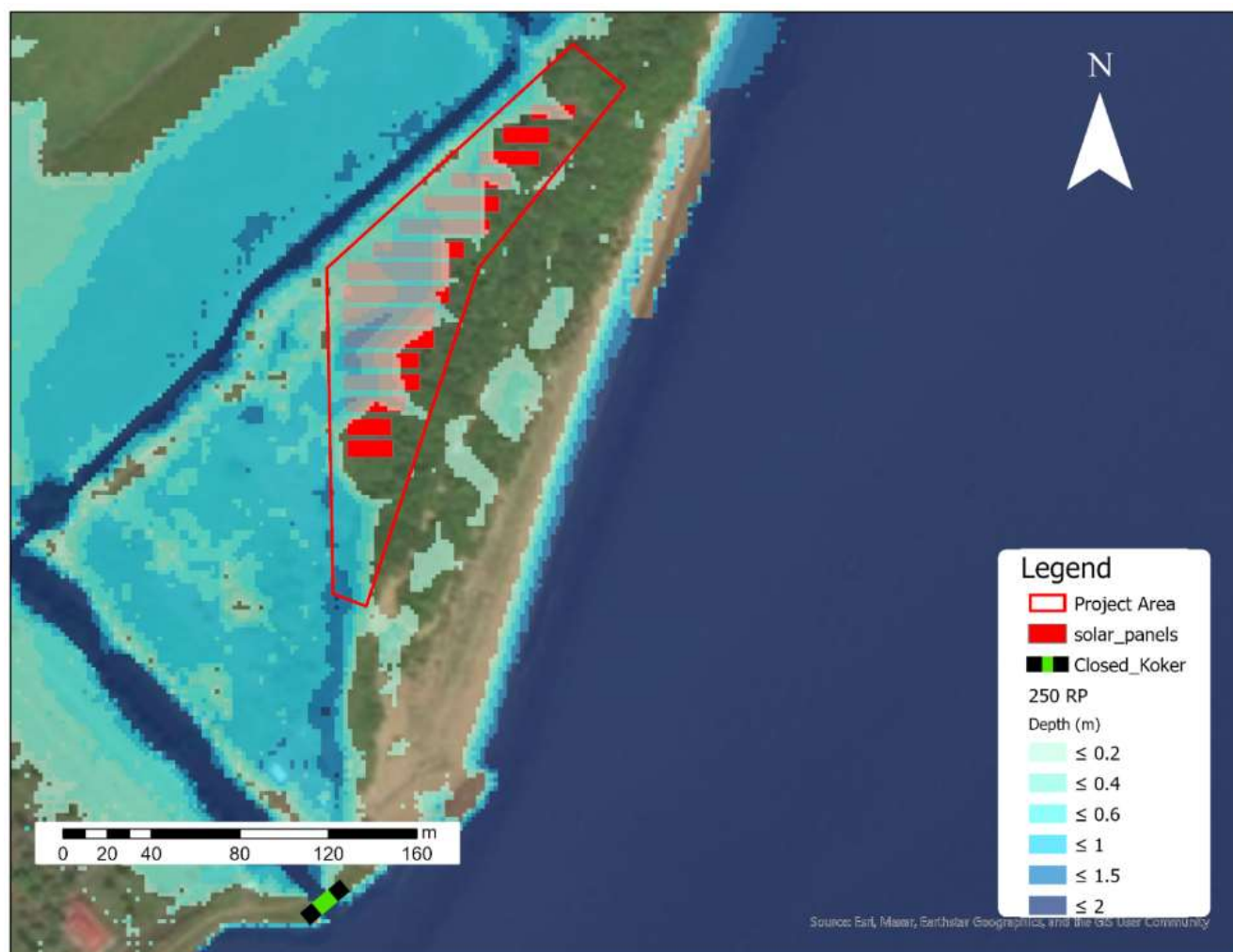


Figure 4.7 Showing site-specific flood level raster for the 250 Yr Rainfall Event where the koker is closed. (HEC -RAS 6.0).



Figure 4.8 Showing site-specific flood level raster for the 5 Yr. Rainfall Event where the koker is closed. (HEC -RAS 6.0).

On the other hand, modelling the scenarios wherein the koker was open showed no-real variation between the 5RP and 250 RP events, this indicated that the levels were mainly dependent on the tidal levels applied at the mouth of Essequibo. From this scenario, maximum depths of 1.09 – 1.12m of inundation were observed on the lowest point on the site (+0.4m).

Table 4.4 Showing inundation depths and their associated return periods.

RP	Future Climate Gates Closed Depth (m)	Future Climate Gate Opened depths (m)
5-Year	0.28	1.09
10-Year	0.29	1.09
25-Year	0.32	1.10
50-Year	0.33	1.11
100-Year	0.53	1.11
250 - Year	0.78	1.12

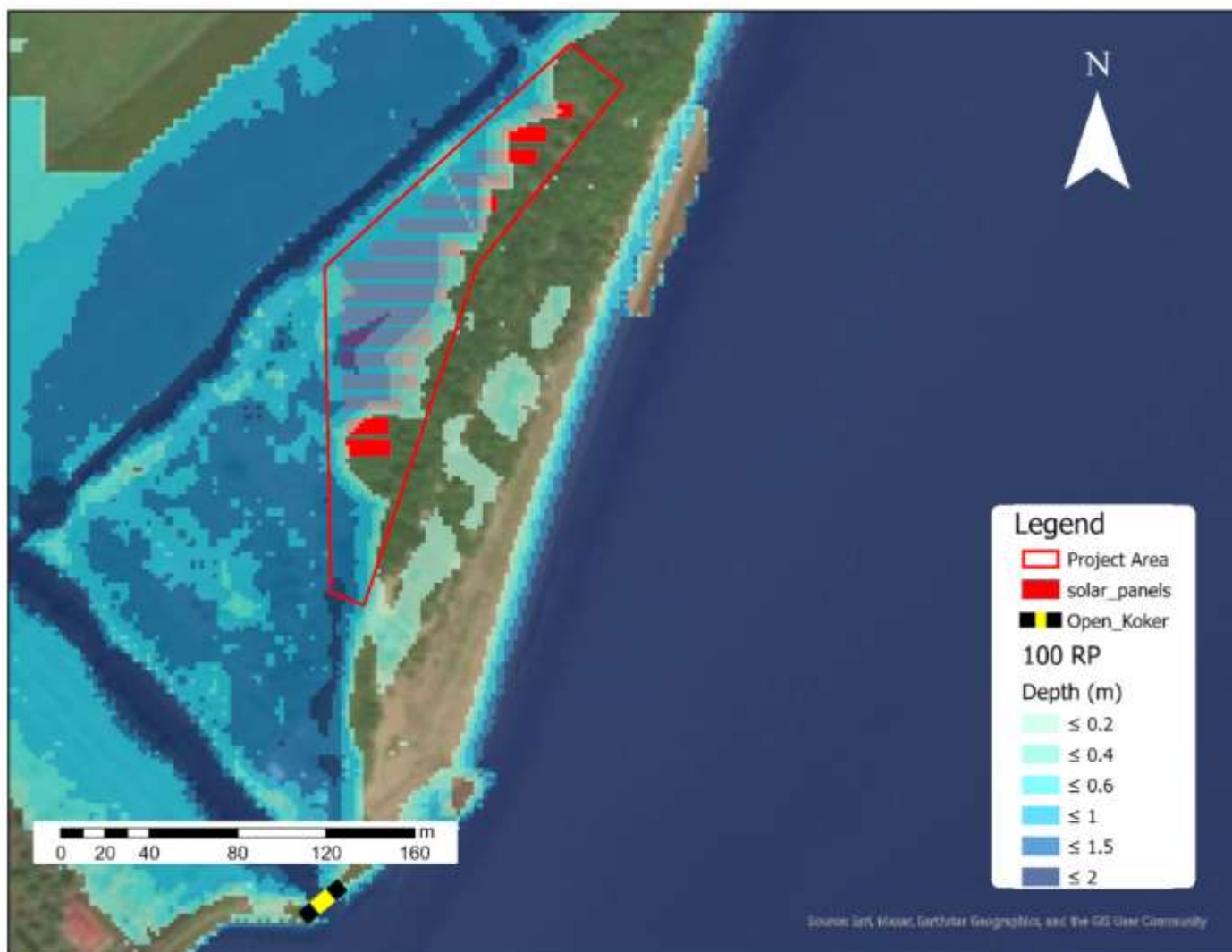


Figure 4.9 Showing site-specific flood level raster for the 100 Yr. Rainfall Event where the koker is closed. (HEC -RAS 6.0).

As previously mentioned, one of the bases for the selection of precipitation events was the co-occurrence of those events with historical accounts deemed to have the most significant flood and damage impacts on areas at the mouth of the Essequibo. These events typically saw high volumes of precipitation in Georgetown and Leguan while upstream stations only experienced a much smaller fraction of increase and not more than a 5RP event.

From the modelled result increases in water level due to significant increases in upstream flows were minimal. Observations of Essequibo discharges indicated that there were increases in flows as a result of the return period, however, the increases in volumes were not of a significant enough magnitude to drastically raise water levels over the expansive capacity of the Essequibo. The only real variance inflows were simply the rate at which the events achieved their peak, observations saw higher RPs achieving rates of increase but still plateauing in and around the same region. The small increases in water level meant that the major increases seen were a result of the tide, hence the general consistency between return periods.

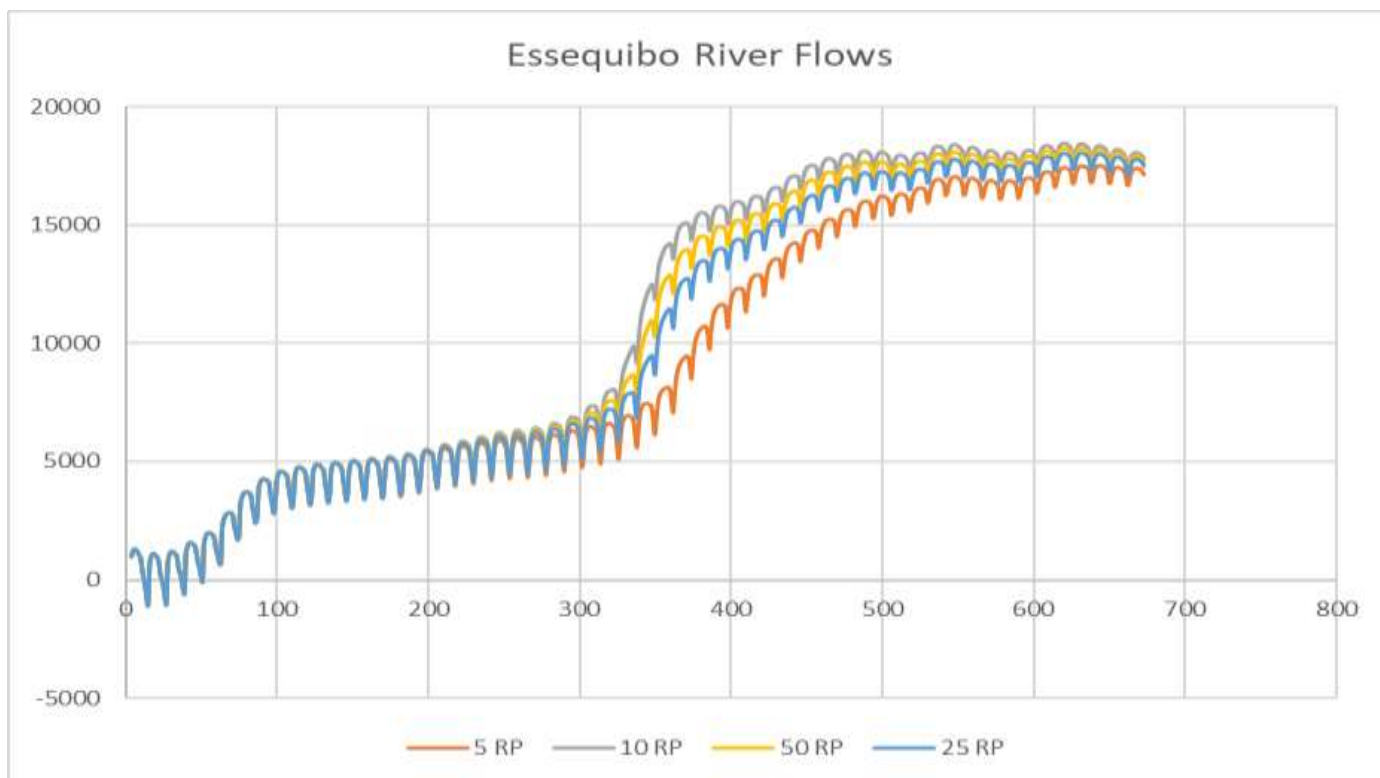


Figure 4.10 Flow Hydrograph of Essequibo River (m^3/s) 5 km upstream of Leguan for 5-25 RP event.

Table 4.5 Showing Water surface elevation and associated flows as a function of the return period at the river cross-section adjacent to the project site.

RP	Max Water Surface Elevation (m)	Flow at Upstream point of Leguan (cms)
5-Year	1.49	17537
10-Year	1.49	17756
25-Year	1.50	18048
50-Year	1.51	18248
100-Year	1.51	18450
250 - Year	1.52	18739

4.1.4 Summary & Recommendations

The most significant concern to the project area is the water level variation due to tidal impacts. Under typical operating scenarios and keen management of the koker systems inundations due to tides should be mitigated to a maximum depth of 0.8m above ground level. However, in instances of failure depths may increase up to 1.1m. To alleviate threats to equipment, it is recommended that site filling operations or raised platform and framing systems be utilized to mitigate against submersion or contact with electrical components. As such the appointed development team should ensure that minimum site elevations or equipment bases exceed 1.2m above means sea level for a 250-year RP event. However, for optimal protection, it is recommended that the levels exceed 1.5m in the event of tidal protection failure.

4.2 Sea Level Rise

Rises in localized sea levels are based on thermal expansion and salinity, both affected by increases in temperature. Increasing temperatures naturally warm the oceans and affects salinity by adding fresh water to the ocean through the melting of glaciers and ice sheets, thus causing sea-level rise (SLR). Sea level rates are increasing across the globe at an accelerating pace, especially in the 20th century. The IPCC AR6 report highlighted with high confidence that the global mean sea level has risen faster since 1900 than over any preceding century in at least the last 3000 years. This report also stated that it is virtually certain that the global mean sea level will continue to rise over the 21st century. The average rate of sea-level rise across the globe was 1.3 [0.6 to 2.1] mm/yr between 1901 and 1971, increasing to 1.9 [0.8 to 2.9] mm/yr between 1971 and 2006, and further increasing to 3.7 [3.2 to 4.2] mm/yr between 2006 and 2018 (high confidence). The human influence was very likely the main driver of these increases since at least 1971.

Application of the RCP8.5 SLR scenario based on the IPCC AR5 report. This scenario was chosen because it represents the worst case of all the emissions scenarios regarding the concentration of GHGs in the atmosphere associated with future global development patterns to the end of the century. Projected Sea Level Rise data was derived from the CMIP5 collection and is presented as 1 x 1-degree resolution which was used to determine local projected sea level rise for the project area¹⁰.

Sea Level Rise is expected to permanently alter Guyana's coastline due to its location and coastal characteristics. IPCC projections show SLR increasing by 0.27m by 2050 and 0.75m by 2100. This could pose a greater inundation threat as the low-lying areas become more susceptible to inundation due to higher high tides, especially in lower coastal sections of the Essequibo River.

Table 4.6 Projected Sea Level Rise for the project area (RCP 8.5)

Year	Projected Sea Level Rise (m)
2043 (End of Design Life of project)	0.22
2050	0.25
2100	0.75

¹⁰ <https://climateknowledgeportal.worldbank.org/country/guyana/impacts-sea-level-rise>

4.3 Wave Climate and Storm Surge

4.3.1 Introduction

For coastal areas, it is important to understand the probable conditions that a proposed coastal zone will be subjected to, to provide maximum protection adequately and appropriately against extreme events. Extreme wave climate and storm surge levels are critical in determining any flood risks the site may experience. Extreme waves from hurricanes inform the structural design and storm surge levels while swell waves inform the functional design of the stabilisation of the shoreline.

Hurricane storm surge is an increase in water levels during the passage of a hurricane, which is above the normally expected astronomical tides. The increases are due to several factors which include:

- Wind
- Inverse barometric pressure
- Tides
- Waves
- Bathymetry

Wind-driven surge is produced by the force of strong winds moving cyclonically around a storm and pushing water onto the shore in the direction of its movement. These wind stresses induce wind setup, which is the tendency for water levels to increase at the downwind shore and decrease at the upwind shore. The pressure effects of a tropical cyclone will cause the water level in the open ocean to rise in regions of low atmospheric pressure and fall in regions of high atmospheric pressure.

A storm surge may coincide with normal high tides resulting in storm tides causing extreme flooding in coastal areas. This added elevation creates a passage for waves to form and propagate on top reaching further inland. Surges and wave heights onshore are affected by the bathymetry of the sea floor. A narrow shelf, or one that has a steep drop off from the shoreline, thus producing deep water waves near the shoreline, tends to produce a lower surge but a higher and more powerful wave. Contrastingly, a very wide and shallow slope can produce a greater storm surge than a steep shelf. Therefore, it was crucial to determine the storm surge elevations generated at the project site to set the design parameters for floor levels.

4.3.2 Climate Change Consideration

Consideration was given to the potential effect of climate change on the project area to ensure that the proposal is consistent with the Climate change policy. The climate change variables considered were: sea-level rise (SLR), storm intensities and the associated storm surge.

The projections of the Atlantic hurricane activity have been executed by numerous regional and global climate change models to explore the potential changes in hurricane activities. (Emanuel), (Knutson), (Villarini) and the Intergovernmental Panel on Climate Change (IPCC) 1.5° C Report have conducted studies that aim to predict frequency patterns and intensity of Atlantic and Global Hurricanes.

The Intergovernmental Panel on Climate Change (IPCC) has made projections based on numerical models that indicate tropical storms are far more intense than in previous years. The IPCC 1.5° C Report stated that, under high levels of warming, very intense hurricanes are expected to occur more frequently, although the overall number of hurricanes is expected to reduce.

Knutson (2013)¹¹, stated that with warming occurring in the Atlantic Ocean over the twenty-first century there will be fewer tropical storms and hurricanes overall; there will also be an increase in the frequency of very intense (categories 4 and 5) hurricanes under the representative concentration pathway 4.5. The author stated that this increase is similar with those of [Murakami et al. \(2012\)](#), who used a high-resolution global model, which projected a nonsignificant increase in category 4 and 5 storm days in the Atlantic basin (+15%) and globally (+4%) this is reflected in the future climate conditions used for deep water wave conditions.

¹¹ Knutson, Thomas R., Joseph J. Sirutis, Gabriel A. Vecchi, Stephen Garner, Ming Zhao, Hyeong-Seog Kim, Morris Bender, Robert E. Tuleya, Isaac M. Held, and Gabriele Villarini. "Dynamical downscaling projections of twenty-first-century Atlantic hurricane activity: CMIP3 and CMIP5 model-based scenarios." *Journal of Climate* 26, no. 17 (2013): 6591-6617.

4.3.3 Hurricane Wave Hindcast

4.3.3.1 Methodology

Over 24 storms have passed within 200nm of Guyana in the past 170 years. It was necessary to define the deep-water hurricane wave climate at a point offshore the project area, to establish safe floor levels above the anticipated storm surge. The offshore point from which hurricane track data within a 400 km radius passed is shown below:

- Latitude: 9.608530 North
- Longitude: - 56.89946 West



Figure 4.11 Historical hurricane tracks (left) within 200nm of Guyana (NOAA) and location of offshore point (right) used for Extremal analysis, showing the track used in the analysis

The National Hurricane Centre (NOAA) database of hurricane track data in the Caribbean Sea was utilised to carry out a hindcast, followed by a statistical analysis to determine the hurricane waves and wind setup conditions. The database of hurricanes, dating back to 1886, was searched for storms that passed within a 400km radius of the site. The following procedure was carried out.

1. Extraction of storms and storm parameters from the historical database. A historical database of storms was searched for all storms passing within a search radius of 400km radius of the site.
2. Application of the JONSWAP Wind-Wave Model. A wave model was used to determine the wave conditions generated at the site due to the rotating hurricane wind field. This is a widely applied model and has been used for numerous engineering problems. The model computes the wave height from a parametric formulation of the hurricane wind field.
3. Application of Extremal Statistics. Here the predicted maximum wave height from each hurricane was arranged in descending order and each was assigned an exceedance probability by Weibull's distribution.
4. A bathymetric profile from deep water to the site was then defined and each hurricane wave was transformed along the profile. The wave height at the nearshore end of the profile was then extracted from the model and stored in a database. All the returned nearshore values were then subjected to an Extremal Statistical analysis and assigned exceedance probabilities with a Weibull distribution.

4.3.3.2 Results

A total of 9 storms passed within 400km of the site for the period 1862 to 2017 (155 years), 3 of the storms were classified as cat 4-5, however, the intensity of the storms when passing Guyana passed at category not surpassing category 3. For coastal areas, it is important to understand the probable conditions that a proposed coastal structure will be subjected to, the most frequent hurricane waves from the **North Easterly** direction correlating with the dominant wind direction. The directions mentioned above are more prevalent for the node considered because of the unobstructed path (fetch) for waves to propagate and reach shore. The annual maximum values of extreme wave heights were fitted to a Generalised type III Extreme Value (GEV) distribution to determine the wave heights.

Overall, these are relatively large waves with the potential to cause severe damage along the shoreline. They are, however, deep-water waves that will be impacted by the bathymetry as they approach the shoreline. Therefore, their potential for nearshore wave climates was investigated using a wave refraction and diffraction model, highlighted in Section 5.3.3.1

able 4.7 Summary of present Wave Height predictions at the site from the NE Direction.

Return period (year)	Present Wave height, HS (m)	Future Wave height, HS (m)	Period (seconds)	Dominant Direction
----------------------	-----------------------------	----------------------------	------------------	--------------------

5	5.1	5.1	11.2	North- East
10	5.4	5.4	11.6	
25	5.7	5.7	11.9	
50	5.9	6.1	12.1	
100	6.1	7.3	12.3	
250	6.3	8.4	12.5	

4.3.4 Deepwater Swells Hindcast

The swell wave climate was also necessary to define the impact of the nearshore waves on the project site's shoreline. The deep-water wave heights and periods were extracted from a global wave projection model that ran wave climate simulations using a 1-degree global implementation of WaveWatch III (v3.14). Results of the simulations were published as network common data form (NetCDF) files by The Commonwealth Scientific and Industrial Research Organisation (CSIRO) based in Australia. For the purposes of this project, the data was extracted for the Representative Concentration Pathway (RCP) 8.5 trajectory for the mid-century period (2026 – 2045). The trajectory was chosen as it best simulates our current trajectory of increased gas emissions and population growth through the end of the century with nominal policies to reduce emissions. An offshore point was then selected approximately 50 km from the shoreline at:

- Latitude: 9.608530 North
- Longitude: - 56.89946 West

The deep-water wave data was then used to generate bi-variant tables for the mean wave heights versus periods. The swell waves were estimated by taking the highest 99th percent exceedance waves (12-hour wave) from the bi-variant table (see Appendix). The swell wave heights are of a magnitude of 2.6m with a period of 9.3 seconds in deep water, the data deduced was used in the nearshore wave model.

Table 4.8 The wave height (meters) and corresponding wave period (seconds) for the future (2041–2060) swell waves

Swell Wave	
Wave Height (m)	2.6
Period (seconds)	9.3
Direction	NE

4.3.5 Nearshore Wave Climate and Hydrodynamics

Nearshore wave climate is crucial for assessing coastal processes and their effects on the natural and man-made structures along a shoreline. This wave condition is derived from deep-water wave parameters since offshore waves translate to nearshore waves as they approach the shore. The direct transformation of such a large amount of wave observations is not feasible; thus, advanced techniques using numerical wave models are used to quickly drive nearshore wave conditions from offshore wave data. Current and future wave climate scenarios were simulated in Mike for 5-100 return period for deep-water hurricane waves and swell waves.

Model Description: Mike 21/3 Couple Model

The MIKE 21/3 Coupled FM Module suite of computer programs was used to calculate the corresponding distribution of surface water elevation and waves in the area. MIKE 21/3 Coupled Model FM is a truly dynamic modelling system for application within coastal, estuaries, and river environments. When using the suite, it is possible to simulate the mutual interaction between waves and currents using a dynamic coupling between the Hydrodynamic Module and the Spectral Wave Module. The two (2) modules are employed as:

- The hydrodynamic module calculates the solution for the surface elevation and velocity field at each point in the domain as a function of time with a critical Courant–Friedrichs–Levy (CFL) number.
- The spectral wave module to model the wave propagation and transformation from offshore up to the shoreline was calculated using the spectral wave component

The hydrodynamic model simulates water level variation and flows in response to a variety of forcing functions in lakes, rivers, estuaries and coastal regions. The hydrodynamic module can be used to solve both three-dimensional and two-dimensional problems. The discretization of the governing equation in geographical and spectral space is performed using the cell-cantered finite volume method. In the geographical domain, an unstructured mesh

technique is used. The time integration is performed using a fractional step approach where a multi-sequence explicit method is applied for the propagation of wave action.

4.3.5.1 Model Development

Finite Element Mesh Development, the process of mesh development, entails the following steps:

- The input of bathymetric data for the wider area and in detail for the project area
- Specifying of vertices/nodes in the mesh
- Element construction in the mesh
- Interpolation for depth at vertices/nodes
- Specifying open boundaries and land

The mesh constructed for the calibration and existing configuration extended 16 kilometers in a north-south direction and 14 kilometers in an east-west direction. The outer deep-water areas were gridded with large mesh which gradually decreases on approach to the project area, in keeping with the Courant Number criterion for numerical stability. The eastern and western boundaries were used as open boundaries on which waves and tides were applied.

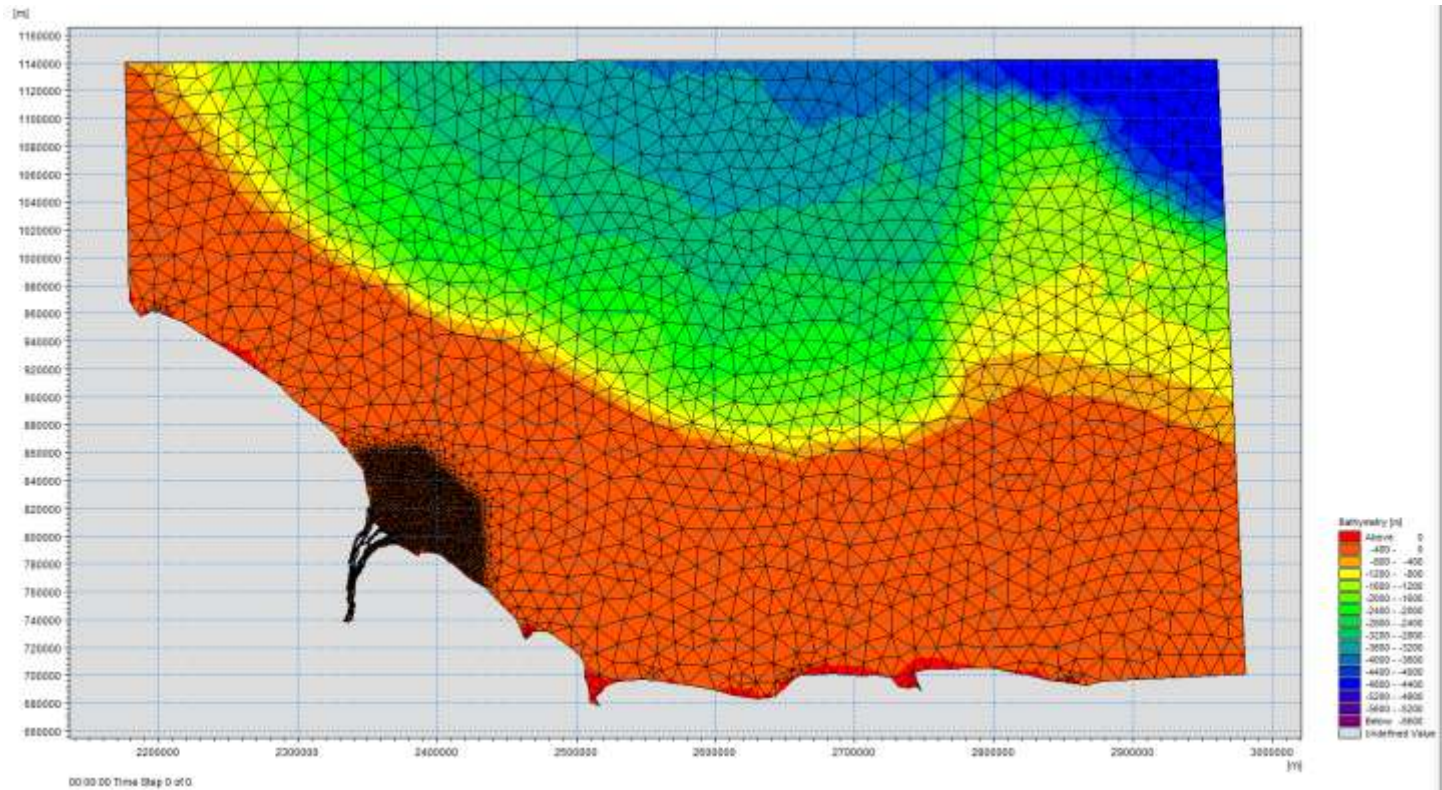


Figure 4.12 Mesh used for modelling of operational and swell scenarios

4.3.5.2 Modeling Scenarios

The following scenarios were executed to evaluate the vulnerability of the shoreline within the project area. The scenarios are described below:

1. Swell waves Present Scenario: The scenario was necessary to describe the damage the infrequent, high-energy waves would have on the beach. Swell wave conditions are generally infrequent and occur a few days out of the year. The waves are fairly large (2.6m wave heights) and have long periods which enables them to cause significant damage to beaches and other structures near the shoreline. It was, therefore, important to look at the swell wave climate to understand the impact on the existing and proposed shoreline and also to design shoreline protective structures, which can withstand these scenarios. The model was used to simulate swell wave conditions from offshore to nearshore for waves approaching the site from the North-East, North and East-North-East directions.
2. Future (2041 – 2060) Hurricane and Swells climate: The rate of climate change globally within the next century is expected to be significantly higher than it was in the past. Trends observed in historical and current climate data are analyzed used to project future climate. Scientists have predicted that there will be fewer storm events but with greater intensities. This scenario is needed to evaluate how resilient the shoreline is and what changes are needed to make it more resilient to future climate change.

3. Storm Surge: to establish a basis to describe the storm surge conditions to understand the present climate, as well as, to predict potential future changes to come.

4.3.5.3 Model Inputs

The data present in Table 4.3 represents the environmental input in the nearshore hydrodynamic model to emulate the natural conditions the site may experience. These inputs were held consists throughout the various scenarios to determine the worst-case events that could occur.

Table 4.9 Environment input for nearshore

Component	Magnitud e	Unit	Dominant Direction	Comment
Wind	4	m/s	NE	Average Annual Wind Speed
Tide Water Level relative to MSL	+/- 1.5	m		High/Low tide
Sea Level Rise in 2050	0.25	m		5mm/year
Swell Wave Height	2.6	m	NE	
Hurricane Deep Water Waves	5.1 – 7.3	m	N-NNE	(5-100 present Return periods)
River Discharge	7000	m ³ /s		From Fluvial flood model
Water Temperature	26.	°c		(Mean annual temperature)

4.3.5.4 Summary and Discussion

4.3.5.4.1 Swell Waves

The modelling results showed that swell waves in the present climate are of an average height of approximately 0.2m and 0.22m, respectively, in the nearshore area. It must be noted that wave heights are more significant at the northern-Easterly section of Leguan Island.

Table 4.10 Swell wave heights (m) at the existing shoreline

Swell Waves			
Directions	N	NE	ENE
Significant wave height	0.20m	0.22m	0.21m

4.3.5.4.2 Hurricane Waves

A wave transformation analysis was done to observe how the wave changes as it moves from deep-water to the shoreline. The nearshore wave heights were identified, arriving from three (3) dominant directions namely: North (N), North-Easterly (NE), and East-North-Easterly (ENE) directions, for a 5,10-, 25-, 50- and 100-year return period storm. The models were simulated using both the current and future climate scenarios to better understand the worst-case impact of the waves on the study area as it relates to climate change. The analysis revealed that under the present climate for the 100-year Return Period the wave heights at the shoreline at the site were 0.35m, 0.41m and 0.41m for the N, NE and ENE respectively. The present climate extreme waves at the project site average 0.4m with the NE being the worst-case scenario modelled the 100 Return period storm shown in Figure 4.12.

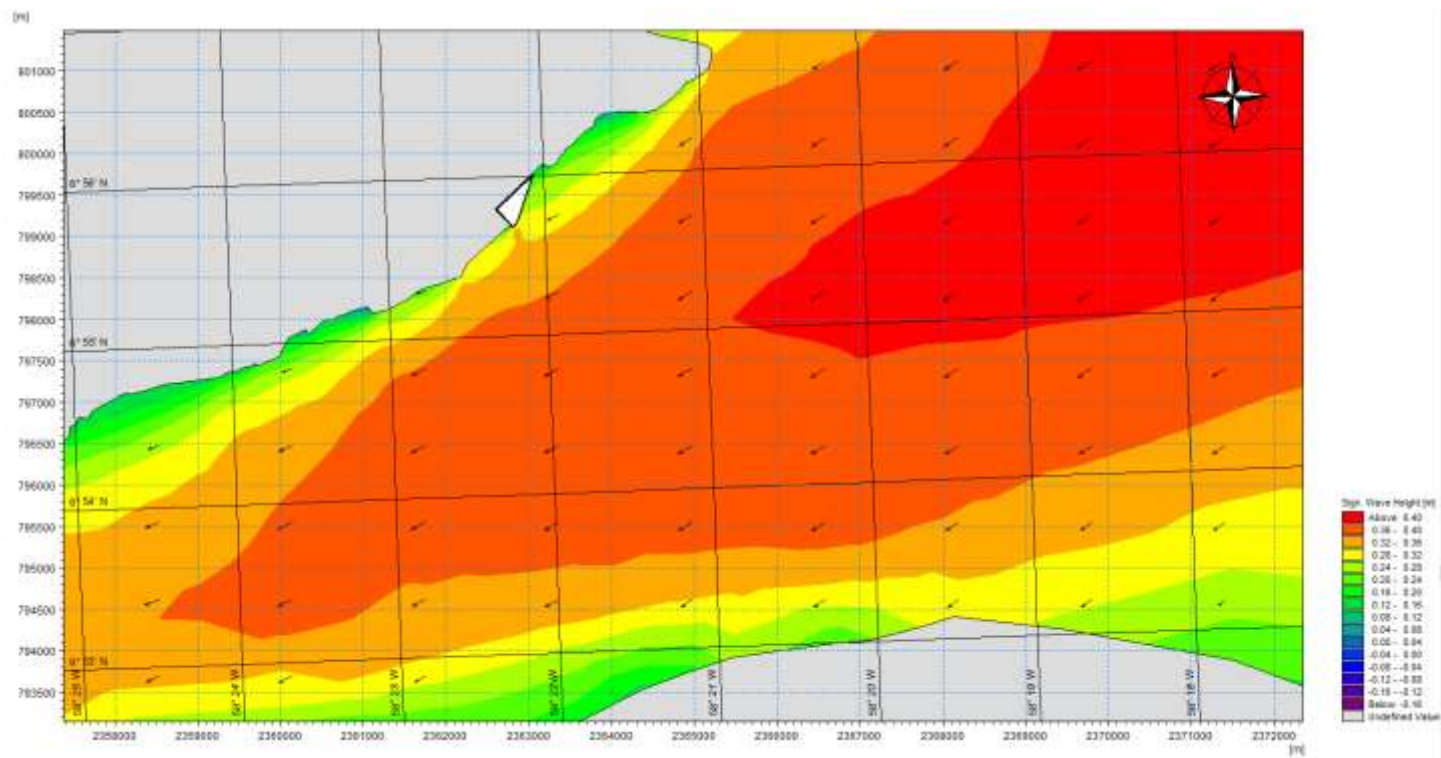


Figure 4.13 Hurricane waves affecting the existing shoreline for a 100 RP storm

Under hurricane conditions, the northwest (NE) direction poses the greatest threat. It was observed that the project area would be partially inundated due to it low-lying nature of the area therefore it would be recommended that protection measures be undertaken to increase the elevations of the proposed assets.

Table 4.11 Summary of 5yr to 100yr return period Hurricane wave heights arriving at the shoreline based on deep water wave transformation modelling.

Hurricane Wave Return Period	Deepwater wave height (m)	Nearshore wave heights (m)
5 – year	5.1	0.25
10- year	5.4	0.29
25 – year	5.7	0.32
50 – year	6.1	0.35
100 – year	7.3	0.41
250- year	8.4	0.43

4.3.6 Storm Surge Model

Storm surges are a meteorological phenomenon, mostly wind storms that pose a geophysical risk which abruptly inundated low-lying coastal regions. Over the past decades, the direct impact of such hazards as storm surges and extreme waves has resulted in grave environmental degradation and socioeconomic disturbances along Guyana’s coast. These hazards are expected to become more severe in the future because of present and projected sea-level rise and more intense hurricanes.

This section aims to examine the inundation risk of extreme water levels in project areas under climate change. A storm surge analysis was done to observe the rise in the seawater level from the change in the meteorological process such as wind and atmospheric pressure, at the shoreline. The storm surge heights were identified for 5,10-, 25-, 50- and 100-year return period storms. The models simulated the current to better understand the waves' impact on the study area related to climate change. The storm surge is considered to occur during the highest high tide with 0.2 meters of sea level rise in addition to the impacts of the waves and river flow as shown in Figure 4-13.

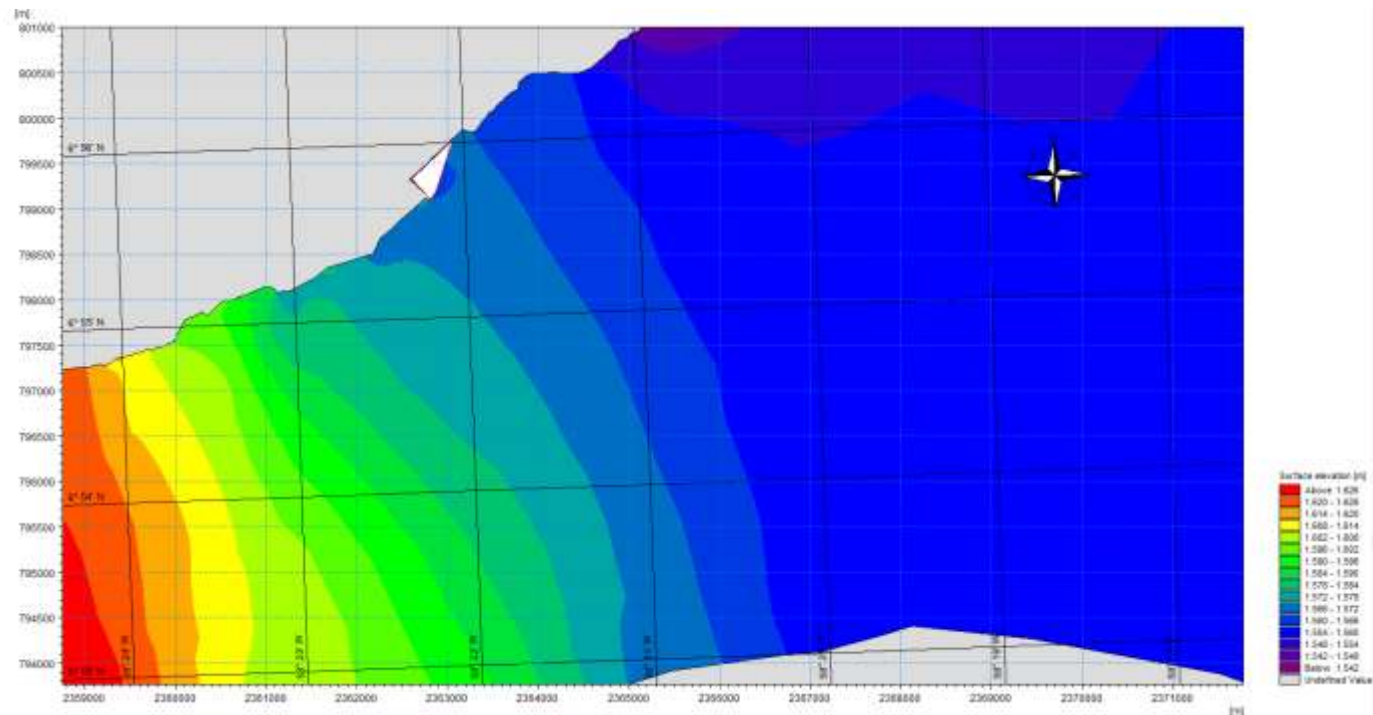


Figure 4.14 100 Yr. Return Period- Present Climate Storm Surge Plot

Storm surge is a relatively rare occurrence in Guyana, as such minimal impact from this phenomenon is expected. It was estimated that the storm surge inundation would cause damage within the project area with inundation depths at the shoreline ranging from 0.2- 0.38m for the 5-yr and 250-yr Return Period storm this caused the surge bypassing the berm and inundate the site. The Current velocities reflect that the river is the driving factor in the hydrodynamics of the project area as the storm surge activities increase the flow of the river decreases, this is summarized in Table 4.12.

Table 4.12 Recommended Floor and Road levels based on 5-yr to 100-yr RP storm surge.

RP	Storm Surge Depths (m) relative to the lowest point in the project area	Current Speed (m/s)
5	0.220	0.38
10	0.24	0.37
25	0.28	0.35
50	0.32	0.33
100	0.36	0.30
250	0.37	0.30

4.3.7 Summary

The Hurricane storm surge is an increase in water levels during the passage of a hurricane, which is above the normally expected astronomical tides. To determine the storm surge levels that may affect the site the existing wave climates were considered, in addition to river flow. Though Guyana is not frequently affected by hurricanes directly, high offshore waves can be generated and pushed into the nearshore area to affect the project area.

Extremal analysis was conducted to determine the offshore wave heights for the 12-hour swell wave heights and wave heights for the hurricane Scenarios. This wave data was used as the boundary condition of the spectral wave models to determine how the offshore waves transform and affect the project site. Generally, the wind and wave climate that affect the project area are mild, however, the extremal analysis is needed to determine the worst-case scenarios that may affect the site, The analysis deduced that the site would be partially inundated by the storm surge under 5 to 100 Yr Return Period. It was estimated that the storm surge inundation would cause damage within the project area with inundation depths at the shoreline ranging from 0.2- 0.36m for the 5-yr and 100-yr Return Period storm.

RP	Future Deep Water wave Heights (m)	Nearshore Water wave Heights (m)	Storm Surge Elevation (m) (High tide +SLR+Storm Surge)	Storm Surge Depths (m)
Swell (12-hour Wave)	2.6	0.2	-	-
5	5.1	0.25	1.92	0.22
10	5.4	0.29	1.94	0.24
25	5.7	0.32	1.98	0.28
50	6.1	0.35	2.02	0.32
100	7.3	0.41	2.06	0.36
250	8.4	0.43	2.07	0.37

4.4 Coastal Erosion

4.4.1 Long term erosion rate

Long-term erosion trends Investigations allow for identifying erosion hot spots and the long-term threats to the project area from retreating shorelines. This was important to identify the actual erosion hotspots that might require stabilization, verify wave transformation modelling, and make provisions for the infrastructure design. Shoreline positions from satellite imagery and aerial imagery were obtained from an unmanned aerial vehicle flight as the primary data sources. The map Figure 4.1 shows the shorelines from satellite imagery for the years 2004 and 2019 highlighting the 60 meters of shoreline accretion occurring over 15 years.



Figure 4.15 Satellite imagery of Historical shoreline overlaid over proposed project area (left 2004 & Right 2019)

The underlying trend of the project shoreline is accretion at an average rate of 2.2m per year along the shoreline in the project area shown in Figure 5.17. This rate of accretion for the site exceeds the expected shoreline loss per year due to sea level rise of 0.28m annually and is projected for 4.7m in the next 25 years if left unprotected. This highlights that the accretion is due to external influence outside of the project area and further analysis is required.

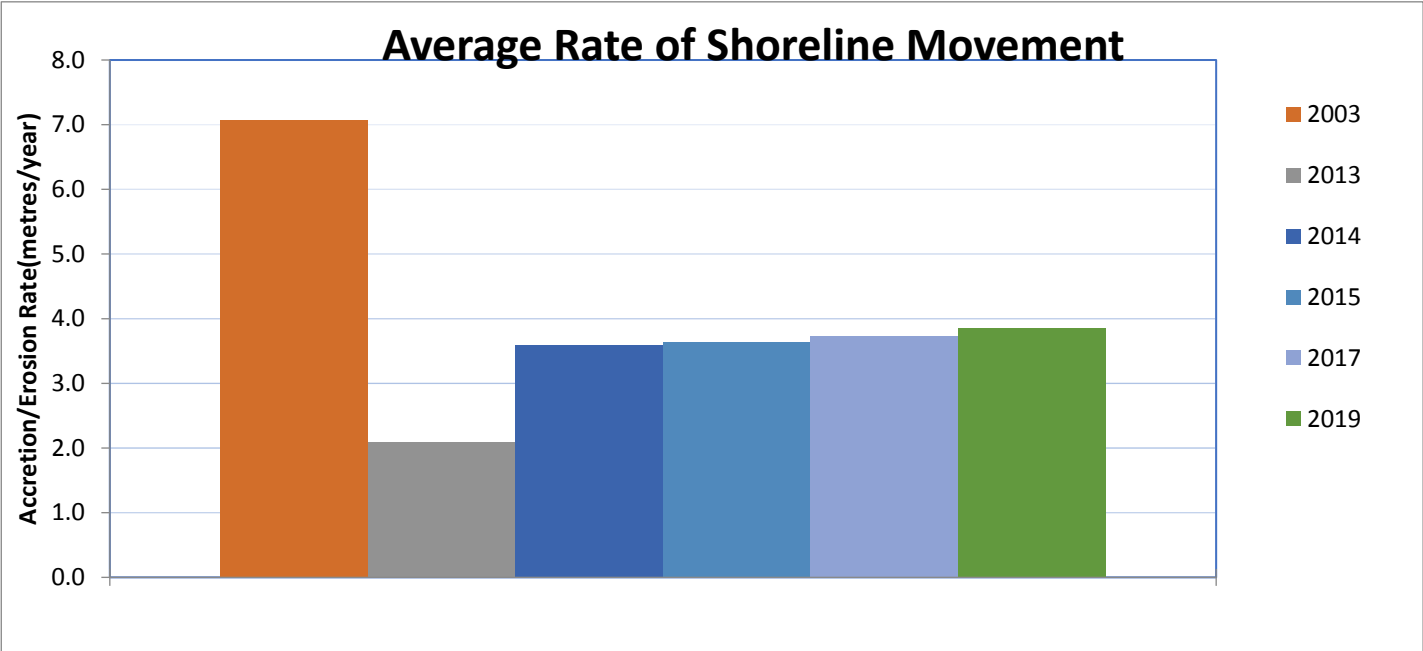


Figure 4.16 Accretion rates for project area for the years 2004 to 2021

4.4.2 Equilibrium Beach Profile

The equilibrium beach profile (EBP) explains the balance between destructive and constructive forces acting on a beach. It gives a quantitative understanding of the characteristics of beach profiles and is central to the assessment of a stable nearshore profile. Sediment samples collected from the shoreline indicate that the shoreline consists of Course sand which an average grain size of 0.66mm. The assessment of the nearshore profile is further complicated by the flow of the Essequibo River moving fine sediments downstream. Grain size analysis, therefore, represents a starting point in the determination of a stable beach profile to determine if the shoreline is stable.

4.4.2.1 Methodology

The methodology employed was as follows:

- 1. Depth of closure was estimated from swell wave climate data and was estimated to be between 4 meters based on the oncoming wave heights.
- 2. The maximum equilibrium beach length of the beach profile is calculated from the reef depth and location
- 3. Grain size for each profile was used to determine the scale parameter A_n . The EBP for the native sand was calculated from the native (D_{n50}) grain size analysis information.
- 4. Bathymetric information for each of the profiles was imported and adjusted from the shoreline.
- 5. An EBP was fitted to the observed profile to determine the scale factor (A_{rms}) that was used to determine the stable sediment grain size D_{n50} if the EBP is reached.

4.4.2.2 Results

The results indicate that the equilibrium profile from the sand samples on the beach is comparable to the EBP generated from the bathymetry (using the least squared method), see Figure 4.16. This would indicate that the shoreline is relatively stable and it is likely that longshore processes are dominant. EBP analysis indicates that the grain size of the native material must be greater than the maximum best-fit grain size of 0.7 mm. It is therefore key to understanding longshore transport rates that affect sediment movement from the project area.

Table 4.13 Native grain sizes of the beach, compared with the grain size of samples measured.

Profile/beaches	Grain size (mm)
Grain size, $D_{n\ 50\ (mm)}$ - Measure d	0.66
Grain size, $D_{n\ 50\ (mm)}$ - EBP	0.7

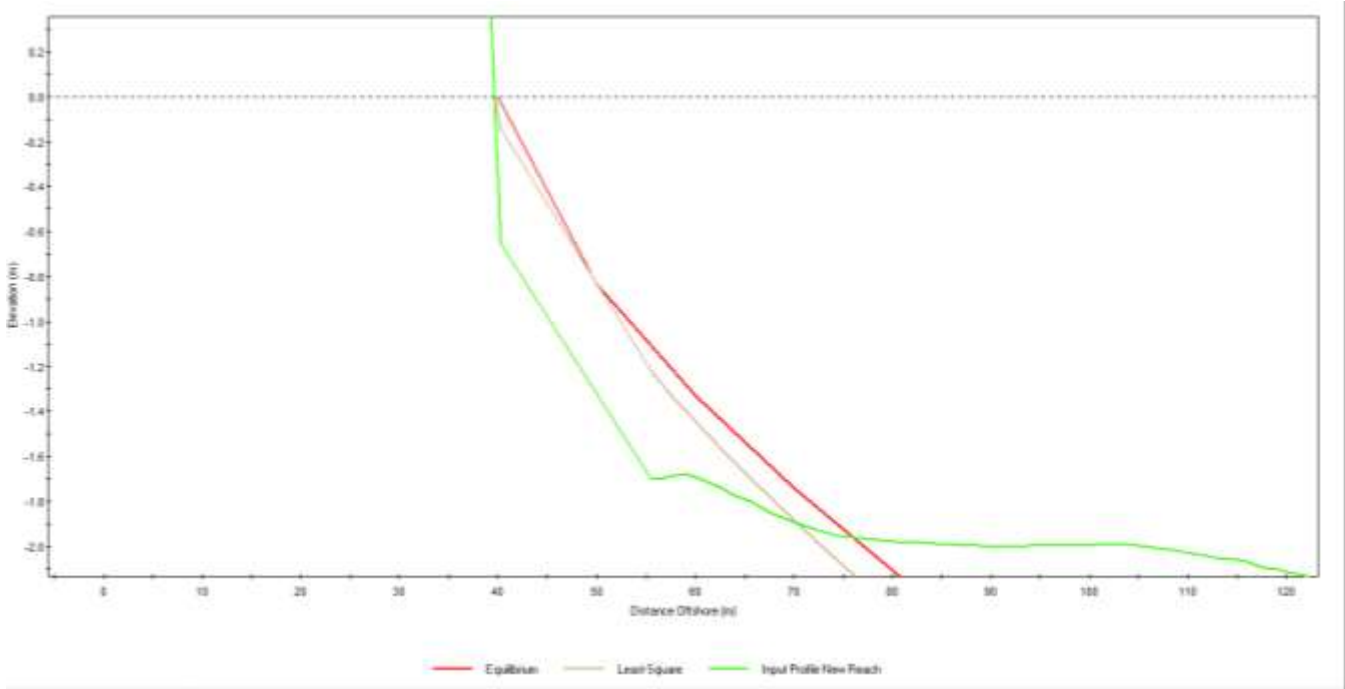


Figure 4.17 Beach profile and RMS curve of Beach

4.4.3 Longshore transport

Longshore sediment transport rates were evaluated to understand better sediment flow directions and transport rates with the project area’s wave climate. The longshore sediment transport rate was estimated using the energy flux method, using the Kamphius, Van Rijn, and CERC to predict volumetric estimates of longshore transport. The longshore transport equations use the operational and swell wave climate over a year to determine the amount of sediment deposited on the respective beach. Wave climate offshore was derived from the deep-water hindcast model and used as the input for these models (*Table 4.5*), along with the beach sand grain size and nearshore profile information.

Table 4.14. Wave climate and occurrence used to drive sediment transport model

Incident Wave Direction	Number of days per year	Ho (m)	Tp (seconds)
N	1	0.3	9.3
NNE	1	0.3	9.3
NE	1	0.3	9.3
ENE	1	0.3	9.3
E	1	0.3	9.3
Oper.	360	0.2	5.5

Longshore transport from both operational and swell waves is responsible for a considerable annual longshore drift of -942m3/year from the shoreline. Operational waves from Northeast are the most dominant driving force for erosion and should be considered in the design of further stabilizing structures. This would be a 0.6m of annual shoreline loss across the 500m of shoreline with a 3m berm.

Table 4.15. Summary of longshore transport rates

Summary		
	Qnet (m³)	Qgross (m³)
Average	942	1217

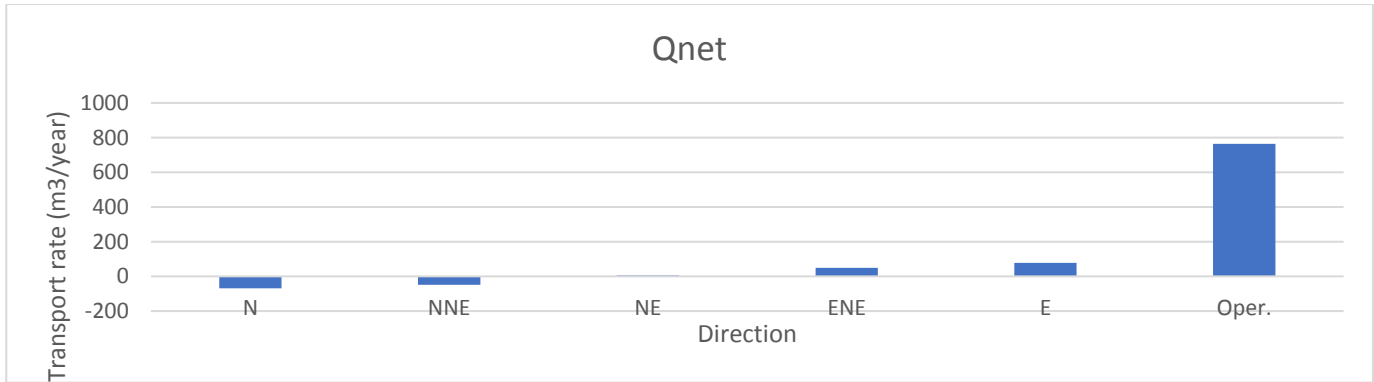


Figure 4.18 Summary of longshore transport rates by direction for the project area

4.4.4 Nearshore Circulation

Nearshore currents are expected to move suspended sediments downdraft and allowed for an estimation of the required length of longshore sediment interrupting structures (groynes) and the width of the surf zone. Longshore currents were determined for the worst-case annual swell from the shoreline to the seaward of the breaker line. The mixing zone length for the swell conditions was determined to be 60m offshore and maximize in water depths of 0.4m. If groynes are utilized the offshore length will have to be 60m in order to be an effective sand trap.

The maximum nearshore mixing velocity is 0.32m/s this is lower than the required scour velocity (0.37m/s) needed to cause the sediment on the beach face to start moving. This would mean the sand on the shoreline is resistant to erosion from the current wave climate. The sediment scour velocity is also higher than the average velocity of the river at 0.3m/s meaning the sand is also stable from the effects of the fluvial action for rainfall return periods less than a 5 rp future event. This analysis indicates that the area is currently relatively stable however shoreline stabilization is required to maintain the current shoreline in the case of high return period events that may cause a lot of damage in a short time.

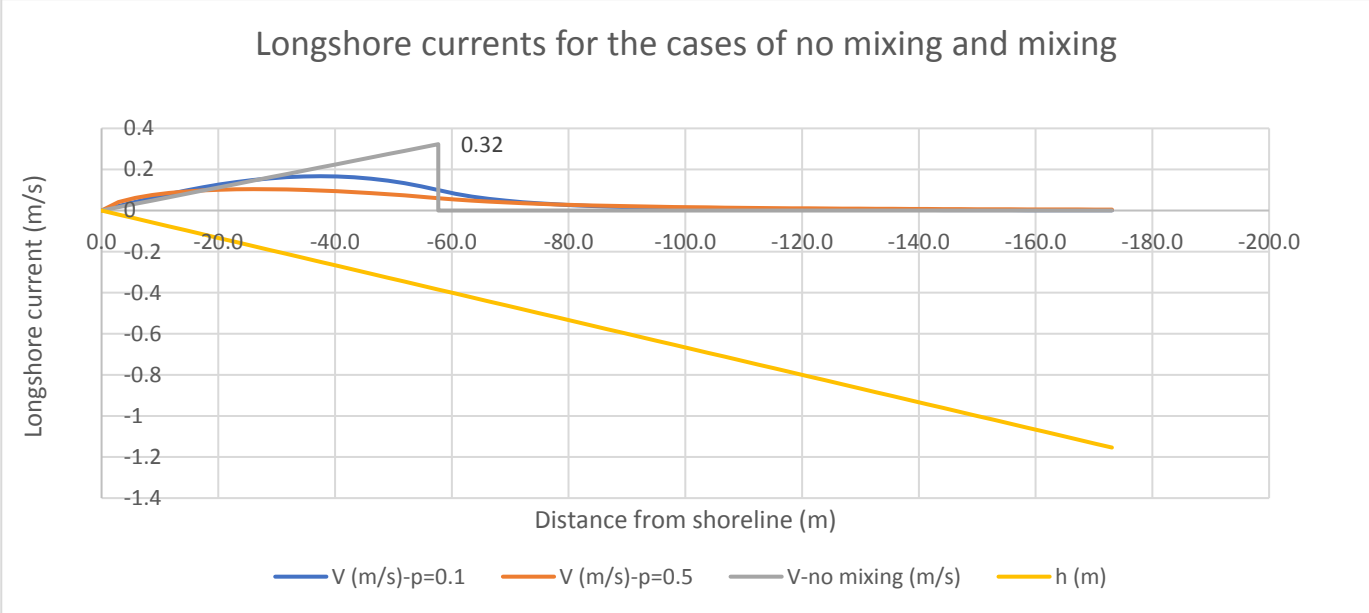


Figure 4.19 Longshore currents for swell waves with existing conditions for Leguan

Table 4.16 Summary of scour and current velocities affecting the project area

Parameter	Value
Scour velocity for native sand (0.66mm) (m/s)	0.37
Near Shore velocity from wave action (swell) (m/s)	0.32
Near Shore velocity from average river flow (m/s)	0.30

4.4.5 Storm induced erosion

It is necessary to determine how the shoreline will respond to the anticipated severe wave climate during hurricane events. The adopted approach was to utilize a cross-shore sediment transport model (SBEACH) to predict the response of the shoreline to waves from design storm events. SBEACH was used to determine the existing shoreline's response to 5-year to 100-year storms from waves approaching the Northeastern (NE) direction.

4.4.5.1 Methodology and data

One profile from the NE direction was cut from land (project site) to deep water. The wave data from the deep-water hurricane model were utilized for this analysis using a 5,10,25, 50 and 100-year return period. Since predicted wave heights were the highest for waves coming from a north-eastern they utilized in the model this is shown in Table 4.18.

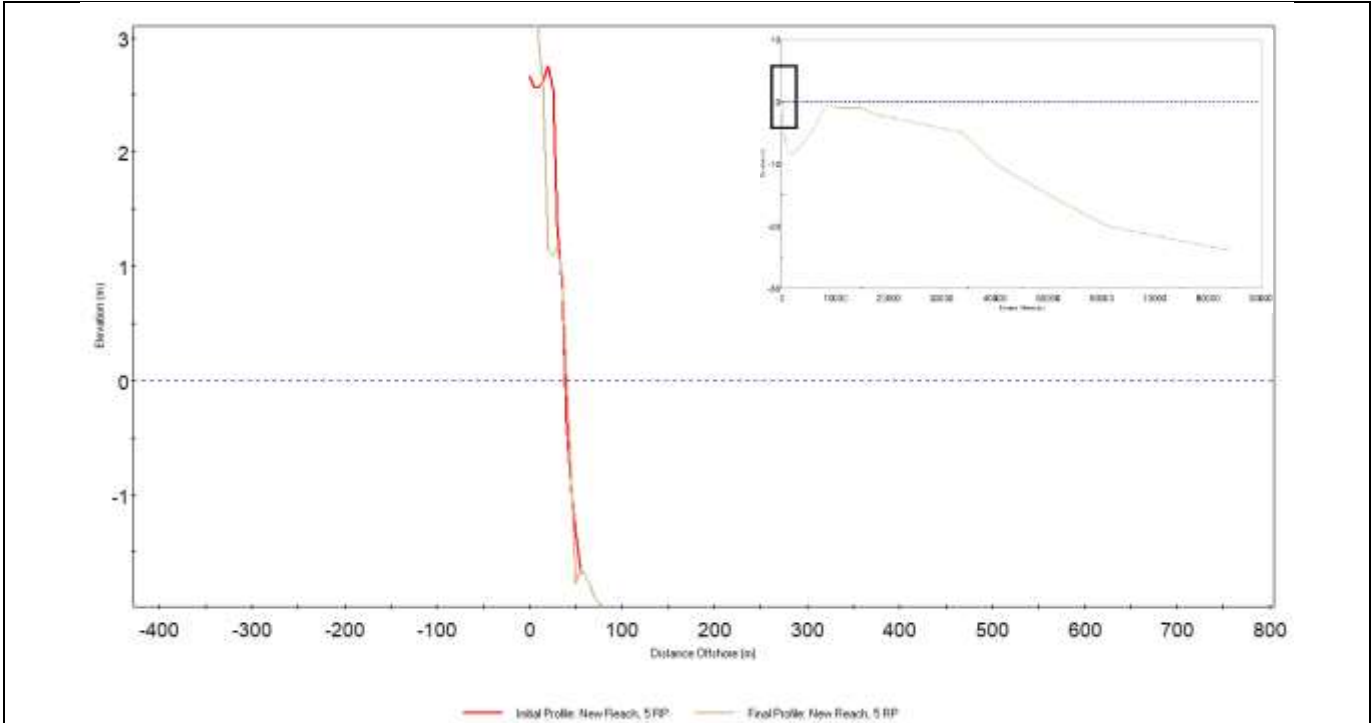
Table 4.17 Input conditions for cross-shore erosion (m) modelling for hurricane wave conditions

Storm	Offshore Future Strom Wave Height, Hs (m)	Period, Tp (s)	Water Elevation (m)
Swell	2.6	9.3	1.7
Hurricane (5- year RP)	5.1	11.2	
Hurricane (10- year RP)	5.4	11.6	
Hurricane (25- year RP)	5.7	11.9	
Hurricane (50- year RP)	6.1	12.1	
Hurricane (100- year RP)	7.3	12.3	
Hurricane (250- year RP)	8.4	12.5	

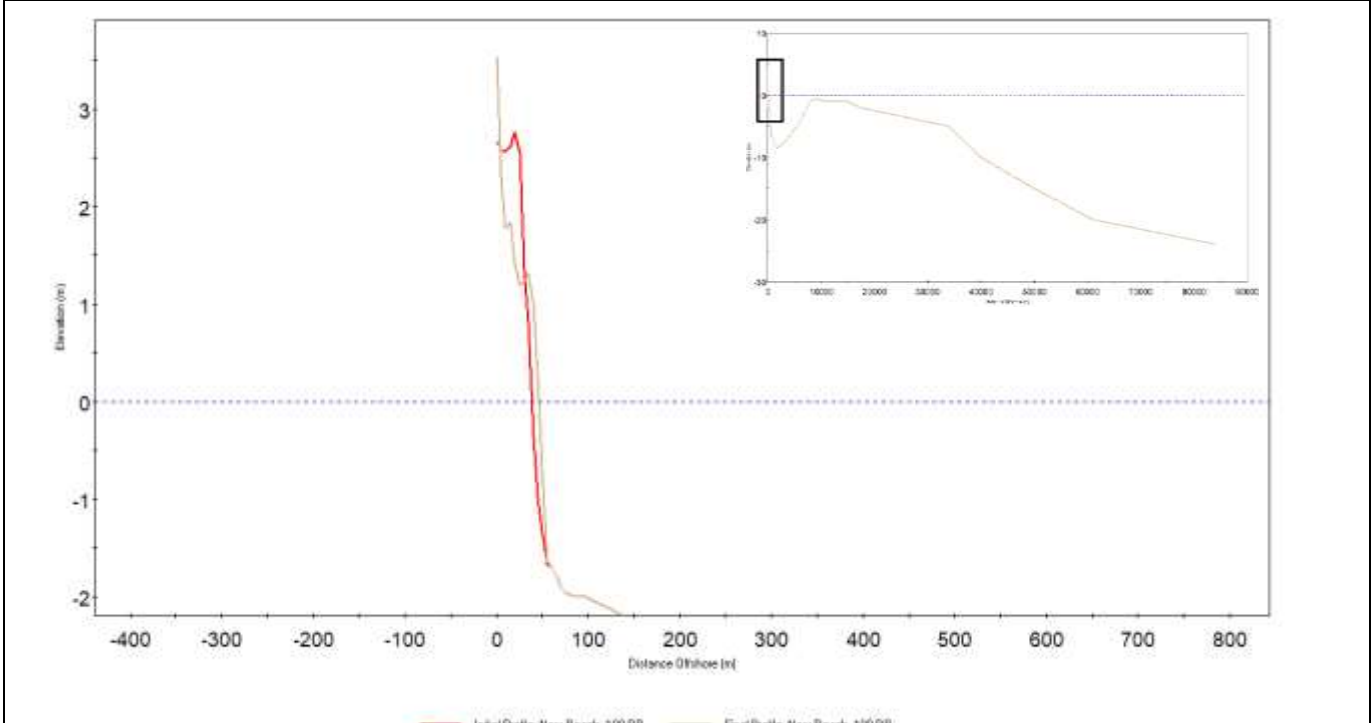
4.4.5.2 Results

The 100 yr event is anticipated to erode the beach face approximately 20m inland and increase the elevation of the berm. Erosion along the profile below can be identified where the initial profile line (red) is above the final profile line (brown), see Table 3.14. The general trend of the scenarios is a landward movement of the shore as the heavy waves push sand up the berm further increasing the elevations in the back of beach area. The berm of the beach as shown in Figure 3.31, Growing on the beach profile and moving the sediments via cross-shore erosion. It is important to note that occurrences of storms in the area are rare, and as such, the extent of erosion will only be experienced during worst case scenarios where a hurricane travels close to the main land. The movement of the shore was predicted to move 8m for the swell wave condition and up to 20m for a 100 RP event.

Table 4.18 Beach Profile showing the extent of shoreline movement after respective storms



Beach Profile showing the extent of shoreline movement after respective after 5 RP future storm



Beach Profile showing the extent of shoreline movement after respective after 100 RP future storm

Table 4.19 showing the Depth and extent of erosion at the project site after swell and storm events.

Wave Direction		Return Period	Max Vertical Erosion ΔZ (m)	Max Horizontal Erosion ΔX (m)
NE	Swell		0.3	8
	Hurricane (5- year RP)		0.3	9
	Hurricane (10- year RP)		0.3	10
	Hurricane (25- year RP)		0.3	12
	Hurricane (50- year RP)		0.6	14
	Hurricane (100- year RP)		0.6	17
	Hurricane (250- year RP)		0.6	20

4.4.6 Summary

The underlying trend of the project shoreline is accretion at an average rate of 2.2m per year along the shoreline in the project area. The results indicate that the equilibrium profile from the sand samples on the beach is comparable to the EBP generated from the bathymetry (using the least squared method). This would indicate that the shoreline is relatively stable, and it is likely that longshore processes are dominant. EBP analysis indicates that the grain size of the native material must be greater than the maximum best-fit grain size of 0.7 mm which is similar to the native sand of 0.66 mm.

Longshore transport from both operational and swell waves is responsible for a considerable annual longshore drift of -942m³/year from the shoreline from N-NE wave directions. Operational waves from North East are the most dominant driving force for erosion and should be considered in the design of further stabilizing structures. This would be a 0.6m of shoreline annual loss across the 500m shoreline with a 3m berm.

The maximum nearshore mixing velocity is 0.32m/s this is lower than the required scour velocity (0.37m/s) needed to cause the sediment on the beach face to start moving. This would mean the sand on the shoreline is resistant to erosion from the current swell wave climate. The sediment scour velocity is also higher than the average velocity of the river at 0.3m/s meaning the sand is also stable from the effects of the fluvial action for rainfall return periods less than a 5 RP future event. This analysis indicates that the area is currently relatively stable however shoreline stabilization is required to maintain the current shoreline in the case of high return period events that may cause a lot of damage in a short time.

The project site is susceptible to short-term erosion with erosion due to storm-induced erosion ranging from 8m to 20m for the 5 and 250 RP storms, respectively. The model predicted that the future site conditions, when simulated against the storm events, would experience erosion of the beach face and the sediment made into a berm. Lastly, 100yr RP storm events produce the most significant landward erosion with 20m inland. It is important to note that occurrences of storms in the area are rare, and as such, the extent of erosion will only be experienced during a worst-case scenario where a hurricane travels close to the mainland.

4.5 Summary

The typical findings indicated that the most significant concern to the project area is the water level variation due to tidal impacts. Under typical operating scenarios and keen management of the koker systems inundations due to tides should be mitigated to a maximum depth of 0.8m. However, in instances of failure depths may increase up to 1.1m. To alleviate threats to equipment, it is recommended that site filling operations or raised platform and framing systems be utilized to mitigate against submersion or contact with electrical components. As such the appointed development team should ensure that minimum site elevations or equipment bases exceed 1.2m above means sea level for a 250-year RP event. However, for optimal protection, it is recommended that the levels exceed 1.5m in the event of tidal protection failure.

RP	Max Water Surface Elevation (m)	Flow at the Upstream point of Leguan (CMS)
5-Year	1.49	17537
10-Year	1.49	17756
25-Year	1.50	18048
50-Year	1.51	18248
100-Year	1.51	18450
250 - Year	1.52	18739

Extremal analysis was conducted to determine the offshore wave heights for the 12-hour swell wave heights and wave heights for the hurricane Scenarios. This wave data was used as the boundary condition of the spectral wave models to determine how the offshore waves transform and affect the project site. Generally, the wind and wave climate that affect the project area are mild, however, the extremal analysis is needed to determine the worst-case scenarios that may affect the site. The analysis deduced that the site would be partially inundated by the storm surge under 5 to 250 Yr Return Period. It was estimated that the storm surge inundation would cause damage within the project area with inundation depths at the shoreline ranging from 0.2- 0.37m for the 5-yr and 250-yr Return Period storm.

RP	Future Deep Water wave Heights (m)	Nearshore Water wave Heights (m)	Storm Surge Elevation (m)	Storm Surge Depths (m)
Swell (12 hour Wave)	2.6	0.2	-	-
5	5.1	0.25	0.72	0.22
10	5.4	0.29	0.75	0.24
25	5.7	0.32	0.78	0.28
50	6.1	0.35	0.82	0.32
100	7.3	0.41	0.86	0.36
250	8.4	0.43	0.87	0.37

The project site is susceptible to short-term erosion with erosion due to storm-induced erosion ranging from 8m to 20m for the 5 and 250 RP storms respectively. The model predicted that the future site conditions, when simulated against the storm events, would experience erosion of the beach face and the sediment made into a berm. Lastly, 250yr RP storm events produce the most significant landward erosion with 20m inland. It is important to note that occurrences of storms in the area are rare, and as such, the extent of erosion will only be experienced during a worst-case scenario where a hurricane travels close to the mainland.

Wave Direction	Return Period	Max Vertical Erosion ΔZ (m)	Max Horizontal Erosion ΔX (m)
NE	Swell	0.3	8
	Hurricane (5- year RP)	0.3	9
	Hurricane (10- year RP)	0.3	10
	Hurricane (25- year RP)	0.3	12
	Hurricane (50- year RP)	0.6	14
	Hurricane (100- year RP)	0.6	17
	Hurricane (250- year RP)	0.6	20

5 Risk

In evaluating the feasibility and profitability of undertaking the solar farm, the risk associated with its development will be evaluated. Risk can be defined as the potential loss of life, injury, destroyed or damaged assets which could occur to a system, society or a community in a specific period of time, determined probabilistically as a function of hazard, exposure, vulnerability and capacity (UNDRR, 2017). “Risk” is a forward-looking concept that implies an eventuality of something that can occur. Assessing risk, therefore, means looking at what are the possible events that can occur, quantifying how likely they are to happen and appraising the potential consequences should they occur (Global Assessment Report, 2015). The factors contributing to the risk will include the presence of hazards that directly impact the solar farm, the degree of vulnerability of the assets, as well as their level of exposure.

5.1 Method

As recommended by the Inter-American Development Bank (IDB) in Chapter 6 of their 2019 reference document, ‘Disaster and Climate Change Risk Assessment Methodology for IDB Projects¹²’, a Probabilistic Risk Assessment was used as the foundation for the Risk Analysis. The Simplified Probabilistic Assessment was used for the project, utilizing the return periods for events triggering the hazard (flooding and erosion), and using damage curves and Average annualized loss (AAL) to determine the project's feasibility.

The general methodology used to assess the potential economic impacts resulting from the hazards can also be summarized in the equation below. It is based on the quantitative approach which aims at quantifying risk according to the hazards, vulnerability of assets and amount of exposure of the asset:

Risk = H * V * A

- H** – Hazard, represented as the annual probability of occurrence for 5 to 250 years return period
- V** – Physical vulnerability of the particular element-at-risk (solar panels and BESS System), expressing the degree of damage or probability of complete loss of the elements at risk given the occurrence of hazard event
- A** – Amount of exposed elements at risk, calculated by overlaying hazard scenarios with the elements at risk. This can be expressed in monetary values or as the number of assets at risk of damage, population at risk and so on.



Figure 5.1: Risk component model (IDB, 2019)

The hazards identified for the project are fluvial flooding, storm surge and erosion of the site. These have been represented as either depths of inundation or as distances of shoreline retreat. The vulnerability of the assets has been tied to the physical components of the panels and BESS that make them especially susceptible to water submersion, that is, the electrical components. The degree of damage is generally assumed to increase drastically once the flood waters, based on their RPs, have come in contact with the electrical components of the assets. The exposure of the assets accounts for the monetary losses associated with damage or replacement of the assets. It has however been assumed that for the flooding hazards, all elements are on the same elevation in the same depth of water, and for the erosion, the shoreline retreat affects the elements near the coast more than those further inland. The comprehensive risk of the project will be presented from a financial perspective to be discussed in further detail in Section 6. Values for the average annualized loss (AAL), cost-benefit ratio (CBR) and internal rate of return (IRR) will be used and will indicate the feasibility of development and to what capacity the project is able to operate in, whether with or without the mitigation measures implemented.

¹² Barandiarán, M., Esquivel, M., Lacambra Ayuso, S., Suarez, G., & Zuloaga, D. (2019). Disaster and Climate Change Risk Assessment Methodology for IDB Projects: A Technical Reference Document for IDB Project Teams.

Average Annualized Loss (AAL) can be defined as the expected losses in any given year, averaged over a long period of time¹³. The AAL is the amount of money the Guyana Energy Agency (GEA) would have to set aside each year to offset future losses (equipment and infrastructure damages) affected by flooding and erosion. This was calculated using direct losses (equipment replacement) and indirect losses (loss of revenue from equipment down time). The AAL was calculated using the integral of the loss exceedance curve, which represents the expected losses in any given year, averaged over an extended period of time. In other words, the sum of the area of the exceedance probability intervals times the incremental damage. This loss exceedance curve was calculated from the expected damage from a hazard at the varying return periods with expected losses derived from the infrastructure damage curves and downtime.

The cost-benefit ratio (CBR) is defined as the ratio between the discounted incremental benefits and the discounted incremental costs, calculated at current commercial or accounting discount rates. This indicator should be higher than 1 for a project to be acceptable. The Internal Rate of Return (IRR) is the discount rate at which a project's net present value (NPV) becomes zero. If the IRR exceeds the discount rate, the project generates returns in excess of other investments in the economy and can be considered worthwhile.

The cash flow (CF) for the project will also dictate the benefits and losses to be had. Cash Flow (CF) is the increase or decrease in the amount of money a business, institution, or individual has. In finance, the term is used to describe the amount of cash (currency) that is generated or consumed in each time period. With respect to the solar farm, the cash flow would be revenue gained from the sale of electricity produced from the new solar plant. Any hindrance to the generation of the power, ie. generation loss due to hazards, would represent lost revenue and would account for downtime of the assets to be reflected in the CBR. A basic outline of the methodology used in analyzing the risk of constructing the solar farm is seen in Figure 5-2. This method incorporates the proposed assessment methodology from the IDB while remaining specific to this project.

¹³ World Meteorological Organization. (2014). Quantifying Risk Before Disasters Occur: Hazard Information for Probabilistic Risk Assessment. Source: <https://public.wmo.int/en/resources/bulletin/quantifying-risk-disasters-occur-hazard-information-probabilistic-risk-assessment>

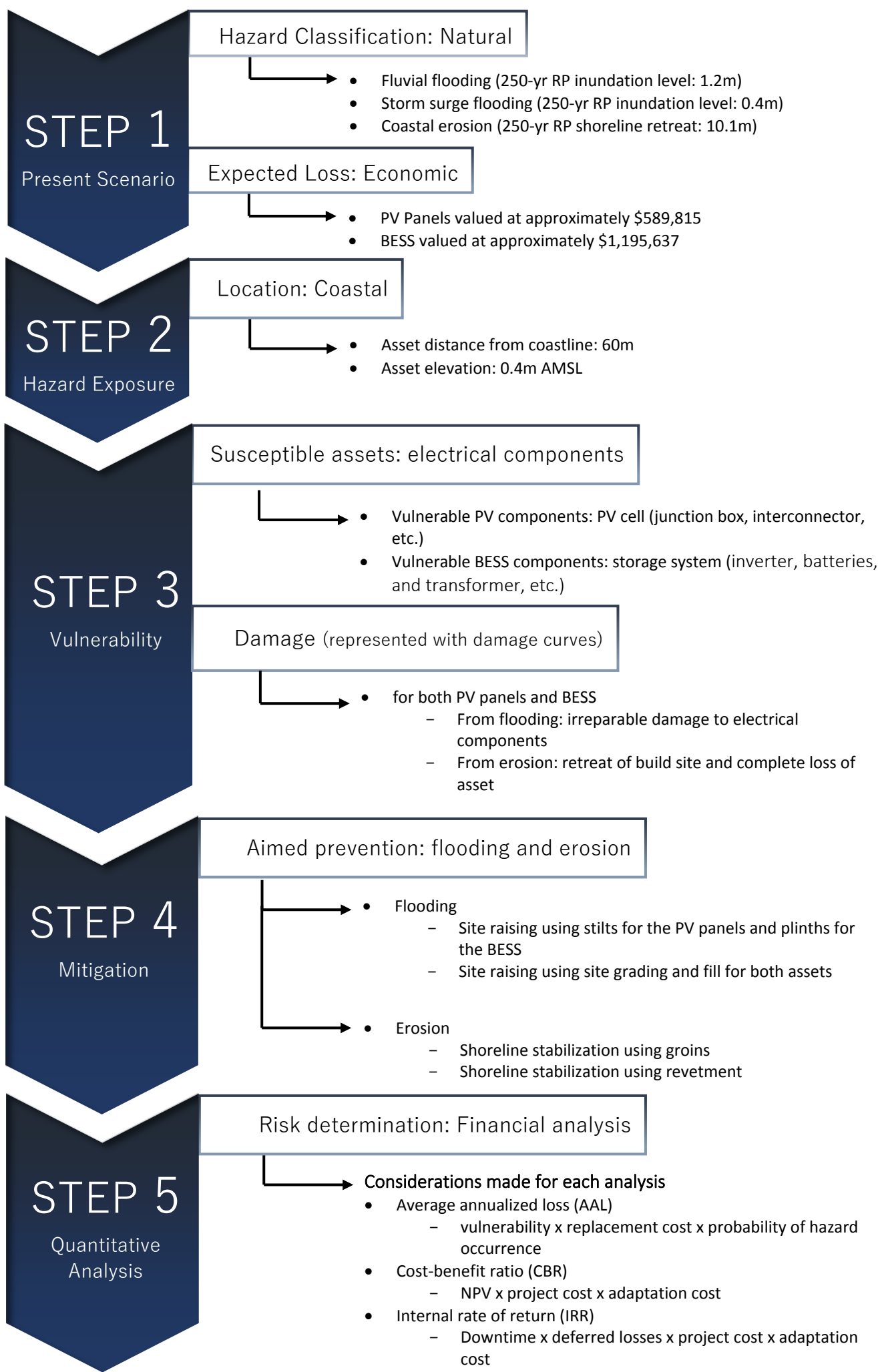
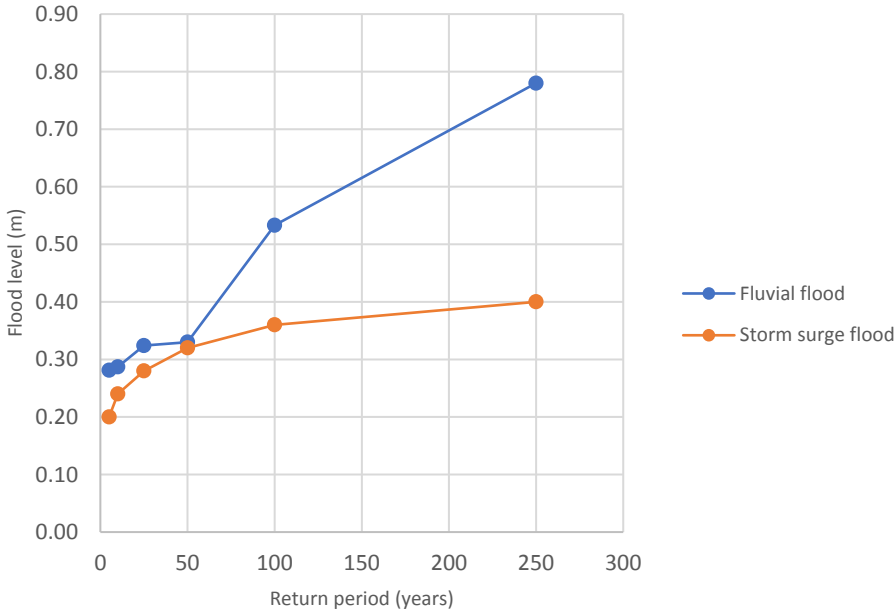


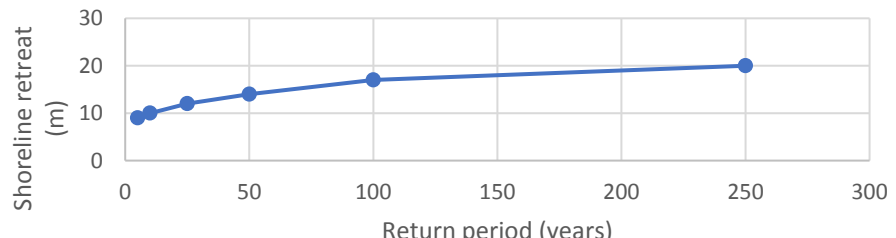
Figure 5.2: Outlined methodology for risk analysis

5.2 Hazards

All hazards were done to show 5 to 250-year return period events. For the fluvial flooding hazard, the HEC-RAS model obtained flood levels as high as 0.8m inundation (above EGL at the Project Site). For the storm surge, the MIKE software obtained storm surge flood levels as high as 0.4m AMSL while for the erosion hazard, the SBEACH software saw erosion (shoreline retreat) as far inland as 60m. A summary of these hazards is seen in Table 5.1: Hazards and expected flooding levels and shoreline retreat for 5 to 250 year RP.

Table 5.1: Hazards and expected flooding levels and shoreline retreat for 5-250 year RP

Hazard		Graph																					
Flooding																							
Due to River		 <table><caption>Flood Level Data (m)</caption><thead><tr><th>Return Period (years)</th><th>Fluvial flood (m)</th><th>Storm surge flood (m)</th></tr></thead><tbody><tr><td>5</td><td>0.28</td><td>0.20</td></tr><tr><td>10</td><td>0.29</td><td>0.24</td></tr><tr><td>25</td><td>0.32</td><td>0.28</td></tr><tr><td>50</td><td>0.33</td><td>0.32</td></tr><tr><td>100</td><td>0.53</td><td>0.36</td></tr><tr><td>250</td><td>0.78</td><td>0.40</td></tr></tbody></table>	Return Period (years)	Fluvial flood (m)	Storm surge flood (m)	5	0.28	0.20	10	0.29	0.24	25	0.32	0.28	50	0.33	0.32	100	0.53	0.36	250	0.78	0.40
Return Period (years)	Fluvial flood (m)		Storm surge flood (m)																				
5	0.28		0.20																				
10	0.29		0.24																				
25	0.32		0.28																				
50	0.33		0.32																				
100	0.53		0.36																				
250	0.78		0.40																				
Return Period	Flood level (m)																						
5	0.28																						
10	0.29																						
25	0.32																						
50	0.33																						
100	0.53																						
250	0.78																						
Due to Storm Surge																							
Return Period	Flood level (m)																						
5	0.20																						
10	0.24																						
25	0.28																						
50	0.32																						
100	0.36																						
250	0.40																						

Erosion (Short-Term)																
Return Period	Shoreline Retreat (m)	 <table><caption>Shoreline Retreat Data (m)</caption><thead><tr><th>Return Period (years)</th><th>Shoreline retreat (m)</th></tr></thead><tbody><tr><td>5</td><td>9</td></tr><tr><td>10</td><td>10</td></tr><tr><td>25</td><td>12</td></tr><tr><td>50</td><td>14</td></tr><tr><td>100</td><td>17</td></tr><tr><td>250</td><td>20</td></tr></tbody></table>	Return Period (years)	Shoreline retreat (m)	5	9	10	10	25	12	50	14	100	17	250	20
Return Period (years)	Shoreline retreat (m)															
5	9															
10	10															
25	12															
50	14															
100	17															
250	20															
5	9															
10	10															
25	12															
50	14															
100	17															
250	20															

5.3 Proposed Mitigation Strategies

To reduce the risk associated with constructing the solar farm, mitigative measures have been proposed that will likely reduce the vulnerability of the assets and decrease the impact that the hazards may have on them. Mitigation will be considered for flood and erosion hazards and the necessary structures put in place.

For fluvial and storm surge flooding, the raising of the assets above the water level to prevent submersion will be proposed. It is important to note that Leguan has sluice gates implemented by the river mouths to control flooding further upstream.

When the flood model was run with a failed sluice gate, the flooding at the site area increased from 0.78m to 1.2m. As a necessary precaution, the assets will be placed above this 1.2m flood water surface. When analysing the profile of the project site, the lowest elevation on the site was found to be about 0.4m AMSL. This will be the reference elevation for all flood water level values and asset elevation values.

As indicated by the GEA, the panels will have two 0.9m high metal supports attached to one side of the panel arrays, and two 2m high metal supports on the other side. With the 1.2m flood level, an additional height of 0.3m will be required for the PV panel system to exist above the flood level and reduce its vulnerability. It is likely that the BESS will only be placed on a 0.3m footing and will require an additional elevation of 0.9m to exist above the flood level. Images of a typical setup up of the panels and BESS can be seen in Figure 5.3..

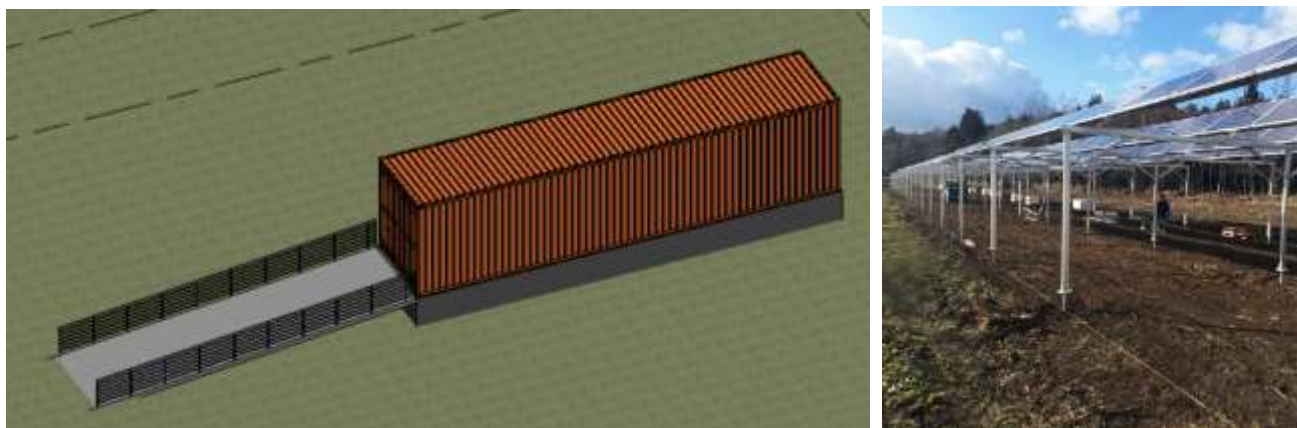


Figure 5.3: 3D rendition of the BESS container (left) and typical solar panel setup on metal supports (right)

As established in Section 1.3, the panels will be assumed to sustain irreversible damage once water contacts the bottom of the PV cells while the BESS is irreversibly damaged once 0.3m of water has flooded inside the container. Metal stilts/posts will be proposed as the mitigative measure to raise the PV panels while concrete plinths will be recommended for raising the BESS. In terms of erosion, this will affect the site as a whole and destabilize the shoreline over time. Hard structures have been considered an acceptable mitigation strategy to combat this hazard; these are groins and revetment. In summary, the mitigative measures can be broken down into 2 hazard scenarios:

1. Scenario 1: Mitigative measures against flooding:
 - a) Scenario 1.1: Stilts/posts and Plinths
 - b) Scenario 1.2: Site grading and fill
2. Scenario 2: Mitigative measures for the site against erosion:
 - a) Scenario 2.1: Groins
 - b) Scenario 2.2: Revetment

The proposed hard structures will be designed with guidance from the Detailed Design and Preparation of Tender Documents of the Works, Supervision and Supply on the Guyana's Sea Defences (9 ACP); a summary report completed in May of 2004 to be referred to hereafter as Guyana's Sea Defences.

5.3.1 Flood Mitigation: Scenario 1

Mitigation measures were considered to reduce the risk of damage for 1.2m flood inundation depth. Since both the Essequibo River and the Atlantic influence, the islands in the estuary at the mouth of the Essequibo River, historic flood events were attributed to either the river or the sea. For all RPs, flood levels attributed to the river were higher than that of the storm surge in the Leguan Island. As a result, mitigation measures will be taken in regard to fluvial flooding.

For the flood mitigations, the costing for construction of stilts/posts and plinths (Scenario 1.1) versus grading and filling the site (Scenario 1.2) was derived. The more cost-effective option was chosen to be used as a part of a finalized mitigation strategy. It should be noted that micro-piles have been added to the cost of raising the PV panels to serve as supports for the panel loads.

Presently, the PV panels are situated on 0.9m stands and the BESS on a 0.3m footing, both relative to the existing ground level (EGL). It is also good to note that the EGL for the site will be at 0.4m AMSL. In the first scenario, concrete plinths will be used to raise the base of the BESS to the same height as the PV panels (0.9m above EGL) inclusive of an access ramp to the container. In the second scenario, the area below the BESS will be filled and graded to the 0.9m elevation (above EGL). A summary table of both scenarios can be seen below.

Table 5.2: Summary of flood mitigation measures for scenarios 1.1 and 1.2

Flood Mitigation - Scenario 1.1 (Stilts/posts and Plinths)						
	Item	Quantity	Unit	Unit Cost (GYD)	Amt (GYD)	Amt (USD)
Solar PV stilts	Hollow sections steel supports (galvanized) 5" x 5" x 4mm	1.2	m	\$4,000.00	\$4,800.00	\$22.97
	Micro Piles	0.25	No	\$20,000.00	\$5,000.00	\$23.92
				addition for labor and transport	\$19,600.00	\$93.78
	Total cost for PV panels				\$29,400,000.00	\$140,669.86
BESS Plinths	8" concrete frame, 2m high, 40' x 8' container (inclusive of footing)	7	m3	\$20,000.00	\$149,230.08	\$714.02
	Ramp/Accessway	16	m3	\$20,000.00	\$320,000.00	\$1,531.10
	Total cost for BESS				\$469,230.08	\$2,245.12
	Grand Total				\$29,869,230.08	\$142,914.98
Flood Mitigation - Scenario 1.2 (Site Grading and Fill)						
	Item	Quantity	Unit	Unit Cost (GYD)	Amt (GYD)	Amt (USD)
Required for Panels	supply and place Backfill and grade site with suitable back fill material to level specified	2430	m3	\$5,000.00	\$12,150,000.00	\$58,133.97
Required for BESS	supply and place Backfill and grade site with suitable back fill material to level specified	26	m3	\$5,000.00	\$133,780.38	\$640.10
	Transportation (2000-ton barge + tug)	4	days		\$51,898,960.00	\$248,320.38
	Labour + Equipment				\$19,302,080.00	\$92,354.45

	Total cost for mitigation measure				\$83,484,820.38	\$399,448.90
SUMMARY						
no	Item	Cost (GYD)		Cost (USD)		
1	Scenario 1.1 (Stilts/posts and Plinths)	\$29,869,230.08		\$142,914.98		
2	Scenario 1.2 (Site Grading and fill)	\$83,484,820.38		\$399,448.90		

Scenario 1.1, using stilts/posts and plinths to raise the panels and BESS, is shown to be the lesser expensive option for mitigating the flood hazard. This scenario will be used when proposing the final mitigation strategy for the site.

5.3.2 Erosion Mitigation: Scenario 2

Erosion mitigation will be done with the aim of ensuring that loss of the shoreline is minimized to the greatest extent. This will be achieved using hard structures along the project site area (for roughly 500m). The structure design will be made from general presumptions taken from the Guyana Sea Defences, and will require more detailed, site-specific analyses.

Though the site experiences mainly accretion, (Reference Figure 2.1), the shorelines are considered unstable, and two (2) erosion mitigation measures have been proposed:

1. The use of Groins (Scenario 2.1)
2. The use of Revetment (Scenario 2.2)

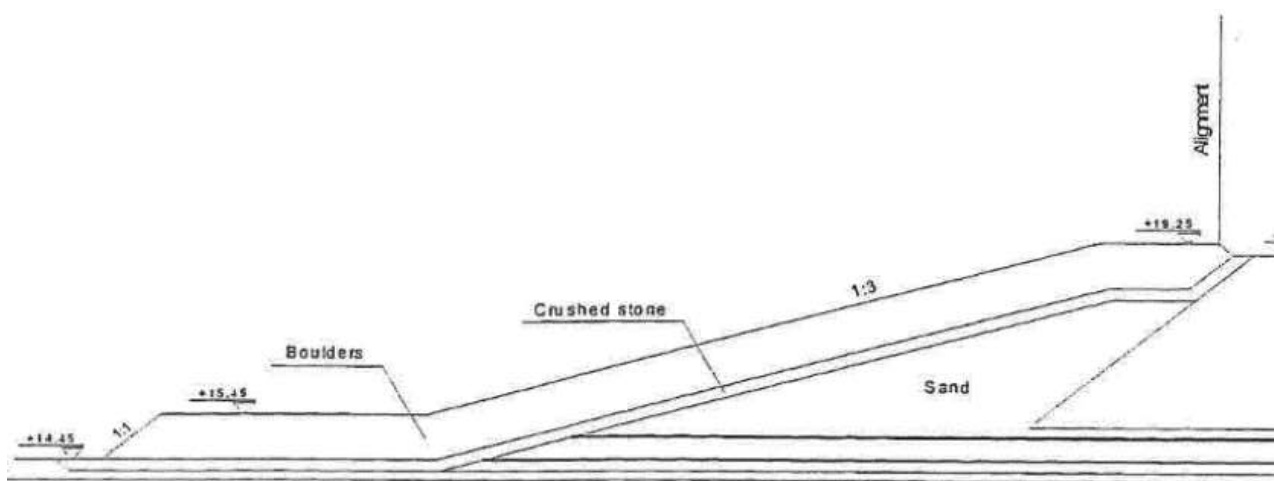


Figure 5.4: Typical section of a riprap standard design revetment (Guyana Sea Defences)

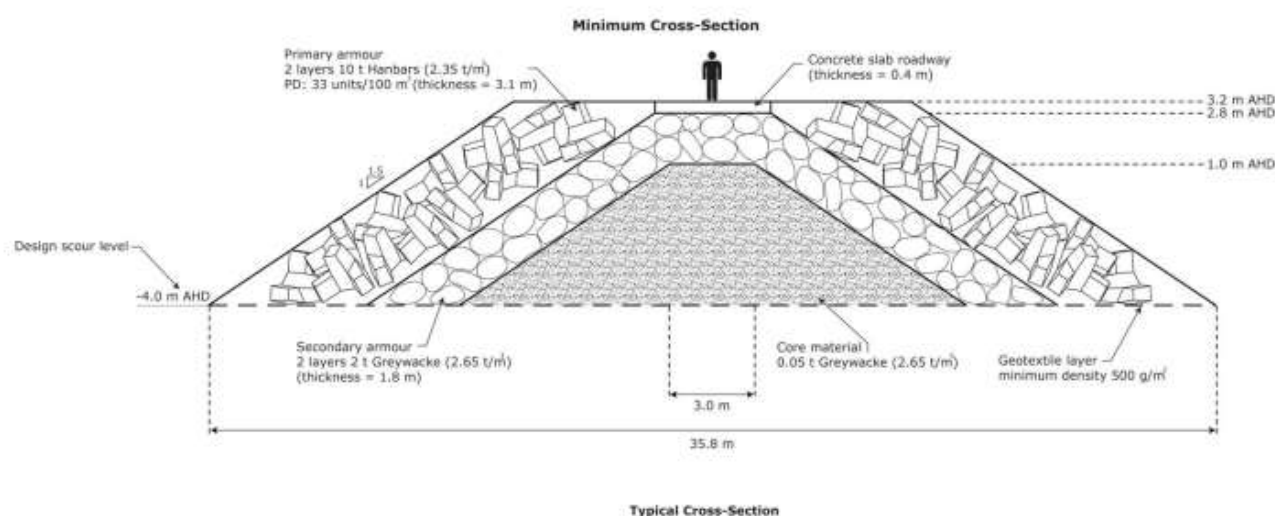


Figure 5.5: Typical section of a groin

In the Scenario 2.1, groins will serve as a barrier to incoming waves and accelerate the accretion of sand by trapping sediments on the up-drift side of the structure. The groins will comprise of thick stones placed on a gravel foundation as suggested by the Guyana Sea Defences (9 ACP) (p. 29-30). In the second scenario, Scenario 2.2, a revetment will both be constructed along the 500m stretch of shoreline by the project site. The revetment will be a riprap standard design (RRSD) with woven geotextiles. These designs and all costing done for both scenarios were also taken from the Guyana Sea Defences (9 ACP). A summary table of both scenarios can be seen below.

Table 5.3: Summary of flood mitigation measures for scenarios 2.1 and 2.2

Erosion Mitigation - Scenario 2.1 (Groins)						
	Item	Quantity	Unit	Unit Cost (GYD)	Amt (GYD)	Amt (USD)
Groin	Groin 6 x t head (83m x 42m ~ 800 m3 each)	6	nr	\$158,840,000	\$953,040,000.	\$4,560,000
Erosion Mitigation – Scenario 2.2 (Revetment)						
	Item	Quantity	Unit	Unit Cost (GYD)	Amt (GYD)	Amt (USD)
Revetment	Riprap standard design, (RRSD) + woven geotextiles (WG)	500	m	\$600,000	\$300,000,000	\$1,435,406
SUMMARY						
No.	Item	Cost (GYD)		Cost (USD)		
1	Scenario 2.1 (Groins)	\$953,040,000		\$4,560,000		
2	Scenario 2.2 (Revetment)	\$300,000,000		\$1,435,407		

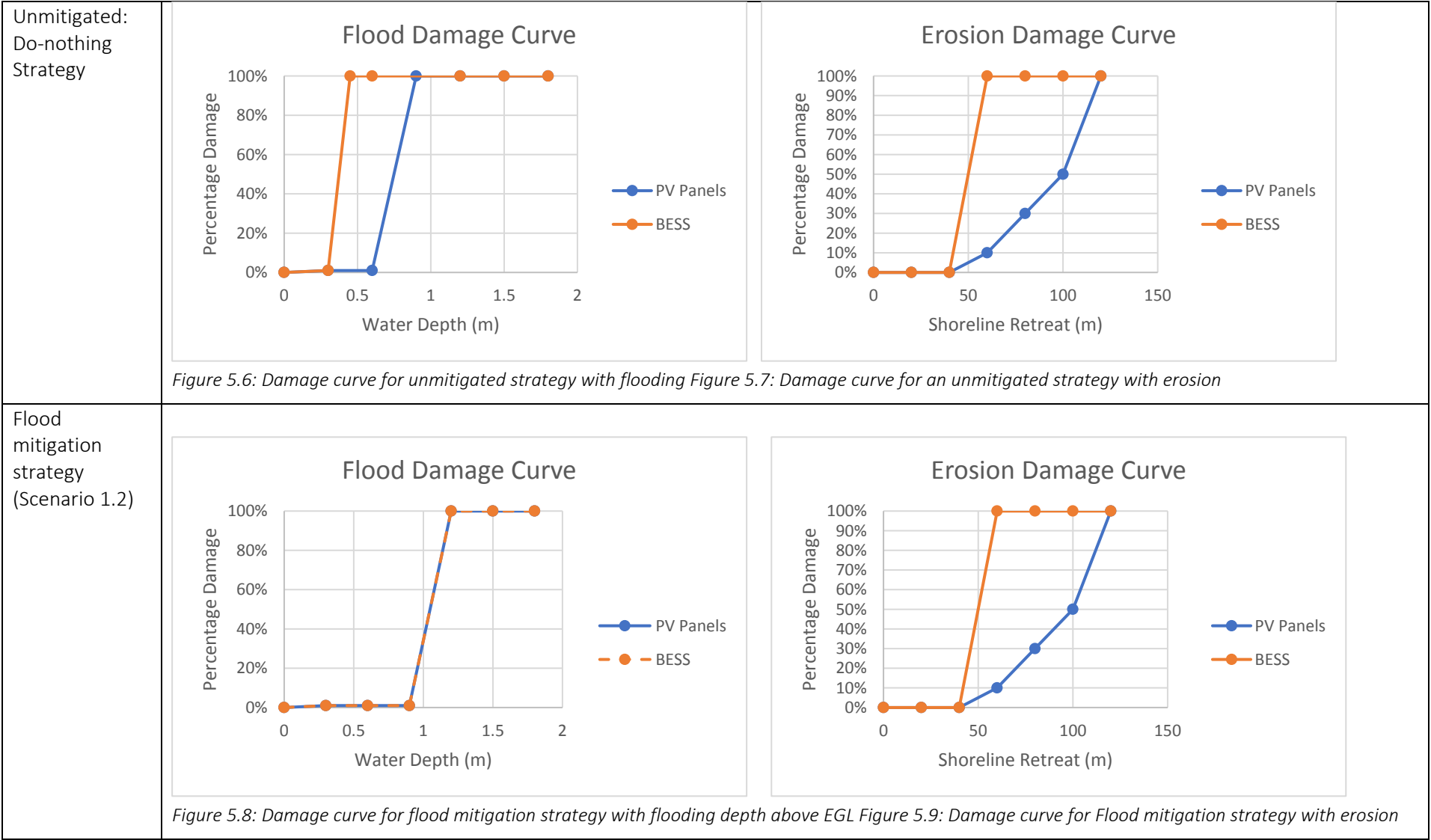
Scenario 2.2, using a revetment to stabilize the shoreline, is shown to be the least expensive option for mitigating erosion. Additionally, when observing the coastal engineering practices of the country, revetment structures are more commonly used than groins. This cultural practice will also be taken into consideration, allowing Scenario 2.2 to be used when proposing the final mitigation strategy for the site.

5.4 Damage Curves

Assets on the project site (PV panels and BESS), while being exposed to the hazards (flooding and erosion), will incur varying levels of damage depending on the depth of inundation (ie. flood level against the asset in relation to the ground). This comparison between the inundation and expected damage level was done for both assets and has been depicted in the figures to follow. The 2 hazards and the respective damage curves produced have been done for the 4 strategies:

- A) Unmitigated strategy (no action)
- B) Flood mitigation strategy (Scenario 1.1: Stilts/posts and Plinths)
- C) Erosion mitigation strategy (Scenario 2.2: Revetment)
- D) Comprehensive Strategy (flooding and erosion protection, Scenario 1.1 + Scenario 2.2)

Damage effects are considered severe and result in the need for complete replacement of the assets once electrical components have been breached as discussed in Section 2.3. For each strategy, it was assumed that replacement of the PV panels and BESS was necessary once water inundation depths exceeded 1.2m. Before those water levels are reached, the property will experience non-detrimental effects such as scouring at the footing. Otherwise, there is no effect or 0% damage. This is true for both the fluvial flooding and storm surge scenarios. For erosion, since the site has a 60m buffer separating it from the coastline, this is the point at which damage effects will begin to be seen. As shoreline retreat increase, more PV arrays are lost, increasing the overall damage percentage. Before the 60 meters, there is no effect.



Erosion mitigation strategy (Scenario 2.2)	<div><h3>Flood Damage Curve</h3><table><tr><th>Water Depth (m)</th><th>PV Panels (%)</th><th>BESS (%)</th></tr><tr><td>0</td><td>0</td><td>0</td></tr><tr><td>0.25</td><td>0</td><td>100</td></tr><tr><td>0.5</td><td>0</td><td>100</td></tr><tr><td>0.75</td><td>100</td><td>100</td></tr><tr><td>1.0</td><td>100</td><td>100</td></tr><tr><td>1.25</td><td>100</td><td>100</td></tr><tr><td>1.5</td><td>100</td><td>100</td></tr><tr><td>1.75</td><td>100</td><td>100</td></tr></table></div> <div><h3>Erosion Damage Curve</h3><table><tr><th>Shoreline Retreat (m)</th><th>PV Panels (%)</th><th>BESS (%)</th></tr><tr><td>0</td><td>0</td><td>0</td></tr><tr><td>20</td><td>0</td><td>0</td></tr><tr><td>40</td><td>0</td><td>0</td></tr><tr><td>60</td><td>0</td><td>0</td></tr><tr><td>80</td><td>100</td><td>100</td></tr></table></div> <div><p>Figure 5.10: Damage curve for erosion mitigation strategy with flooding depth above EGL</p><p>Figure 5.11: Damage curve for erosion mitigation strategy with erosion</p></div>	Water Depth (m)	PV Panels (%)	BESS (%)	0	0	0	0.25	0	100	0.5	0	100	0.75	100	100	1.0	100	100	1.25	100	100	1.5	100	100	1.75	100	100	Shoreline Retreat (m)	PV Panels (%)	BESS (%)	0	0	0	20	0	0	40	0	0	60	0	0	80	100	100
Water Depth (m)	PV Panels (%)	BESS (%)																																												
0	0	0																																												
0.25	0	100																																												
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Shoreline Retreat (m)	PV Panels (%)	BESS (%)																																												
0	0	0																																												
20	0	0																																												
40	0	0																																												
60	0	0																																												
80	100	100																																												
Comprehensive Strategy (Scenario 1.2 + Scenario 2.2): Flood and Erosion Mitigation Strategy	<div><h3>Flood Damage Curve</h3><table><tr><th>Water Depth (m)</th><th>PV Panels (%)</th><th>BESS (%)</th></tr><tr><td>0</td><td>0</td><td>0</td></tr><tr><td>0.25</td><td>0</td><td>0</td></tr><tr><td>0.5</td><td>0</td><td>0</td></tr><tr><td>0.75</td><td>0</td><td>0</td></tr><tr><td>1.0</td><td>100</td><td>100</td></tr><tr><td>1.25</td><td>100</td><td>100</td></tr><tr><td>1.5</td><td>100</td><td>100</td></tr><tr><td>1.75</td><td>100</td><td>100</td></tr></table></div> <div><h3>Erosion Damage Curve</h3><table><tr><th>Shoreline Retreat (m)</th><th>PV Panels (%)</th><th>BESS (%)</th></tr><tr><td>0</td><td>0</td><td>0</td></tr><tr><td>20</td><td>0</td><td>0</td></tr><tr><td>40</td><td>0</td><td>0</td></tr><tr><td>60</td><td>0</td><td>0</td></tr><tr><td>80</td><td>100</td><td>100</td></tr></table></div> <div><p>Figure 5.12: Damage curve for a comprehensive strategy with flooding</p><p>Figure 5.13: Damage curve for the comprehensive strategy with erosion</p></div>	Water Depth (m)	PV Panels (%)	BESS (%)	0	0	0	0.25	0	0	0.5	0	0	0.75	0	0	1.0	100	100	1.25	100	100	1.5	100	100	1.75	100	100	Shoreline Retreat (m)	PV Panels (%)	BESS (%)	0	0	0	20	0	0	40	0	0	60	0	0	80	100	100
Water Depth (m)	PV Panels (%)	BESS (%)																																												
0	0	0																																												
0.25	0	0																																												
0.5	0	0																																												
0.75	0	0																																												
1.0	100	100																																												
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Shoreline Retreat (m)	PV Panels (%)	BESS (%)																																												
0	0	0																																												
20	0	0																																												
40	0	0																																												
60	0	0																																												
80	100	100																																												

5.5 Flood and Erosion vulnerability/ Annualized Losses

Flooding and erosion can cause extensive and almost irreversible damage to infrastructure. The project remains vulnerable to the hazards aforementioned: fluvial flooding, storm surge and erosion. The solar farm can therefore be unmitigated with no shoreline protection or precautionary actions taken for the assets, or the contrapositive done, where measures are put in place. Two (2) mitigation strategies will be considered in reducing the likelihood of coastal retreat. That is hard structures (revetment) as well as measures to protect the assets where a flood event occurs (raising assets' elevations using stilts/posts and plinths). Both the unmitigated and mitigated strategies have been described with the average annual loss (AAL) for each scenario given.

The AAL describes the long-term average loss in value of an asset which can be influenced by conditions that preserve/stagnate/reduce the value of that asset. For the solar farm, the conditions that influence the AAL of the project are the hazards mentioned. In essence, each of the 4 strategies will have the AAL calculated for all 3 hazards.

The AAL will also account for the damages and losses felt by the solar farm whilst out of operation. This will include the downtime for the equipment with the assumption that replacement of the assets after the flood/erosion event, and hence the period of lost revenue, will be 8 weeks. The cost of shutdown of the PV panels and BESS summed to USD\$40,320.00. A further breakdown of the variables used can be seen in the Appendix in Table 8.1.

5.5.1 Unmitigated Scenario

This represents the annualized losses the plant may experience from the hazards described (fluvial flooding, storm surge and shoreline retreat) without any hard structures or asset elevation above the flood level. In order to quantify the risk of such events, annualized losses were calculated for each hazard the project area may experience. The average annualized loss (AAL) is summarized in Table 5.4. In order to reduce the annualized losses several mitigation measures are proposed to reduce downtime and damage to the equipment in the project area. They are summarized in the section that follows.

Table 5.4: AAL (USD) for flood event for unmitigated scenario

Solar PV system (Flood)					
Return Period	Innud. Depth (m)	Vulnerability	Damages + losses	Probability	Expected Loss
5	0.28	0%	\$0	20%	\$0
10	0.29	0%	\$0	10%	\$0
25	0.32	1%	\$5,898	4%	\$177
50	0.33	1%	\$5,898	2%	\$118
100	0.53	1%	\$5,898	1%	\$59
250	0.78	1%	\$5,898	0.4%	\$35
			AAL		\$389
Battery Energy Storage System (Flood)					
Return Period	Innud. Depth (m)	Vulnerability	Damages + losses	Probability	Expected Loss
5	0.28	0%	\$0.00	20%	\$0
10	0.29	0%	\$0.00	10%	\$0
25	0.32	1%	\$11,956.37	4%	\$359
50	0.33	1%	\$11,956.37	2%	\$239
100	0.53	100%	\$1,195,637.14	1%	\$6,038
250	0.78	100%	\$1,195,637.14	0.4%	\$7,174
			AAL		\$13,810
TOTAL AAL			\$14,199		

Table 5.5: AAL for storm surge event for unmitigated scenario

Solar PV system (SS)					
Return Period	Innud. Depth (m)	Vulnerability	Damages + losses	Probability	Expected Loss
5	0.20	0%	\$0	20%	\$0
10	0.24	0%	\$0	10%	\$0
25	0.28	0%	\$0	4%	\$0
50	0.32	1%	\$5,898	2%	\$59
100	0.36	1%	\$5,898	1%	\$59
250	0.40	1%	\$5,898	0.4%	\$35
			AAL		\$153
Battery Energy Storage System (SS)					
Return Period	Innud. Depth (m)	Vulnerability	Damages + losses	Probability	Expected Loss
5	0.20	0%	\$0.00	20%	\$0
10	0.24	0%	\$0.00	10%	\$0
25	0.28	0%	\$0.00	4%	\$0
50	0.32	1%	\$11,956.37	2%	\$120
100	0.36	1%	\$11,956.37	1%	\$120
250	0.40	1%	\$11,956.37	0.4%	\$72
			AAL		\$239
TOTAL AAL			\$392		

Table 5.6: AAL for erosion event for unmitigated scenario

Solar PV system (Erosion)					
Return Period	Shoreline retreat (m)	Vulnerability	Damages + losses	Probability	Expected Loss
5	9.00	0%	\$0	20%	\$0
10	10.00	0%	\$0	10%	\$0
25	12.00	0%	\$0	4%	\$0
50	14.00	0%	\$0	2%	\$0
100	17.00	0%	\$0	1%	\$0
250	20.00	0%	\$0	0.4%	\$0
			AAL		\$0
Battery Energy Storage System (Erosion)					
Return Period	Shoreline retreat (m)	Vulnerability	Damages + losses	Probability	Expected Loss
5	9	0%	\$ -	20%	\$0
10	10	0%	\$ -	10%	\$0
25	12	0%	\$ -	4%	\$0
50	14	0%	\$ -	2%	\$0
100	17	0%	\$ -	1%	\$0
250	20	0%	\$ -	0.4%	\$0
			AAL		\$0
TOTAL AAL			\$0		

5.5.2 Mitigation Scenarios

Several strategies were considered in regard to reducing annual losses from the hazards. They are taken from Section 5.1.2 and the most impactful option or combination of options proposed:

- A) Flood mitigation strategy (Scenario 1.1: stilts/posts and plinths)
- B) Erosion mitigation strategy (Scenario 2.2: revetment)
- C) Comprehensive Strategy (flooding and erosion protection, Scenario 1.1 + Scenario 2.2)
- D) Extended comprehensive Scenario (flooding and erosion protection, Scenario 1.2 + Scenario 2.2)

5.5.2.1 Flood Mitigation Strategy

This option is the least expensive for flood mitigation and is drawn from Scenario 1.1, using stilts and plinths. The total annualized loss from all hazards is summarized below.

Table 5.7: AAL for flood event for Flood mitigation strategy (Scenario 1.1)

Solar PV system (Flood)					
Return Period	Innud. Depth (m)	Vulnerability	Damages + losses	Probability	Expected Loss
5	0.28	0%	\$0	20%	\$0
10	0.29	0%	\$0	10%	\$0
25	0.32	1%	\$5,898	4%	\$177
50	0.33	1%	\$5,898	2%	\$118
100	0.53	1%	\$5,898	1%	\$59
250	0.78	1%	\$5,898	0.4%	\$35
			AAL		\$389
Battery Energy Storage System (Flood)					
Return Period	Innud. Depth (m)	Vulnerability	Damages + losses	Probability	Expected Loss
5	0.28	0%	\$0.00	20%	\$0
10	0.29	0%	\$0.00	10%	\$0
25	0.32	1%	\$11,956.37	4%	\$359
50	0.33	1%	\$11,956.37	2%	\$239
100	0.53	1%	\$11,956.37	1%	\$120
250	0.78	1%	\$11,956.37	0.4%	\$72
			AAL		\$789
TOTAL AAL			\$1,178		

Table 5.8: AAL for storm surge event for mitigation strategy (Scenario 1.1)

Solar PV system (SS)					
Return Period	Innud. Depth (m)	Vulnerability	Damages + losses	Probability	Expected Loss
5	0.20 0.24 0.28 0.32 0.36 0.40	0%	\$0	20%	\$0
10		0%	\$0	10%	\$0
25		0%	\$0	4%	\$0
50		1%	\$5,898	2%	\$59
100		1%	\$5,898	1%	\$59
250		1%	\$5,898	0.4%	\$35
				AAL	
Battery Energy Storage System (SS)					
Return Period	Innud. Depth (m)	Vulnerability	Damages + losses	Probability	Expected Loss
5	0.20 0.24 0.28 0.32 0.36 0.40	0%	\$0.00	20%	\$0
10		0%	\$0.00	10%	\$0
25		0%	\$0.00	4%	\$0
50		1%	\$11,956.37	2%	\$120
100		1%	\$11,956.37	1%	\$120
250		1%	\$11,956.37	0.4%	\$72
				AAL	
TOTAL AAL			\$392		

Table 5.9: AAL for erosion event for Flood mitigation strategy (Scenario 1.1)

Solar PV system (Erosion)					
Return Period	Shoreline retreat (m)	Vulnerability	Damages + losses	Probability	Expected Loss
5	9.00	0%	\$0	20%	\$0
10	10.00	0%	\$0	10%	\$0
25	12.00	0%	\$0	4%	\$0
50	14.00	0%	\$0	2%	\$0
100	17.00	0%	\$0	1%	\$0
250	20.00	0%	\$0	0.4%	\$0
			AAL		\$0
Battery Energy Storage System (Erosion)					
Return Period	Shoreline retreat (m)	Vulnerability	Damages + losses	Probability	Expected Loss
5	9.00	0%	\$0	20%	\$0
10	10.00	0%	\$0	10%	\$0
25	12.00	0%	\$0	4%	\$0
50	14.00	0%	\$0	2%	\$0
100	17.00	0%	\$0	1%	\$0
250	20.00	0%	\$0	0.4%	\$0
			AAL		\$0
TOTAL AAL			\$0		

5.5.2.2 Erosion Mitigation strategy

This option is the least expensive for erosion mitigation and is drawn from Scenario 2.2, using a revetment. The total annualized loss from all hazards is summarized below.

Table 5.10: AAL for flood event for erosion mitigation strategy (Scenario 2.2)

Solar PV system (Flood)					
Return Period	Innud. Depth (m)	Vulnerability	Damages + losses	Probability	Expected Loss
5	0.28	0%	\$0	20%	\$0
10	0.29	0%	\$0	10%	\$0
25	0.32	1%	\$5,898	4%	\$177
50	0.33	1%	\$5,898	2%	\$118
100	0.53	1%	\$5,898	1%	\$59
250	0.78	1%	\$5,898	0.4%	\$35
			AAL		\$389
Battery Energy Storage System (Flood)					
Return Period	Innud. Depth (m)	Vulnerability	Damages + losses	Probability	Expected Loss
5	0.28	0%	\$0	20%	\$0
10	0.29	0%	\$0	10%	\$0
25	0.32	1%	\$11,956.37	4%	\$359
50	0.33	1%	\$11,956.37	2%	\$239
100	0.53	100%	\$1,195,637.14	1%	\$6,038
250	0.78	100%	\$1,195,637.14	0.4%	\$7,174
			AAL		\$13,810
TOTAL AAL			\$14,199		

Table 5.11: AAL for storm surge event for erosion mitigation strategy (Scenario 2.2)

Solar PV system (SS)					
Return Period	Innud. Depth (m)	Vulnerability	Damages + losses	Probability	Expected Loss
5	0.20	0%	\$0	20%	\$0
10	0.24	0%	\$0	10%	\$0
25	0.28	0%	\$0	4%	\$0
50	0.32	1%	\$5,898	2%	\$59
100	0.36	1%	\$5,898	1%	\$59
250	0.40	1%	\$5,898	0.4%	\$35
			AAL		\$153
Battery Energy Storage System (SS)					
Return Period	Innud. Depth (m)	Vulnerability	Damages + losses	Probability	Expected Loss
5	0.20	0%	\$0.00	20%	\$0
10	0.24	0%	\$0.00	10%	\$0
25	0.28	0%	\$0.00	4%	\$0
50	0.32	1%	\$11,956.37	2%	\$120
100	0.36	1%	\$11,956.37	1%	\$120
250	0.40	1%	\$11,956.37	0.4%	\$72
			AAL		\$239
TOTAL AAL			\$392		

Table 5.12: AAL for erosion event for erosion mitigation strategy (Scenario 2.2)

Solar PV system (Erosion)					
Return Period	Shoreline retreat (m)	Vulnerability	Damages + losses	Probability	Expected Loss
5	9.00	0%	\$0	20%	\$0
10	10.00	0%	\$0	10%	\$0
25	12.00	0%	\$0	4%	\$0
50	14.00	0%	\$0	2%	\$0
100	17.00	0%	\$0	1%	\$0
250	20.00	0%	\$0	0.4%	\$0
			AAL		\$0
Battery Energy Storage System (Erosion)					
Return Period	Shoreline retreat (m)	Vulnerability	Damages + losses	Probability	Expected Loss
5	9.00	0%	\$0	20%	\$0
10	10.00	0%	\$0	10%	\$0
25	12.00	0%	\$0	4%	\$0
50	14.00	0%	\$0	2%	\$0
100	17.00	0%	\$0	1%	\$0
250	20.00	0%	\$0	0.4%	\$0
			AAL		\$0
TOTAL AAL			\$0		

5.5.2.3 Comprehensive Flood and Erosion Strategy

This option protects from both flooding and erosion while implementing the least expensive strategies. It utilizes Scenario 1.1 and 2.2 for this approach. For Scenario 1.1, the plinths and stilts/posts cost will remain the same. For scenario 2.2, a revetment will be used to protect the shoreline from wave action and accrete sediments over time on the updrift side of the structure. This will assist in stabilizing the shoreline and reducing the risk of erosion. The armour layer of the revetment will be assumed to follow the existing coastal defence design in Leguan, using 700mm granite rock, and extend along the 500m shoreline of the project site. This strategy is costlier than the first due to the inclusion of the revetment structure which had its construction rate taken from Guyana's Sea Defences (9 ACP).

Table 5.13: AAL for flood event for comprehensive strategy (Scenario 1.1 + 2.2)

Solar PV system (Flood)					
Return Period	Innud. Depth (m)	Vulnerability	Damages + losses	Probability	Expected Loss
5	0.28 0.29 0.32 0.33 0.53 0.78	0%	\$0	0.2	\$0
10		0%	\$0	0.1	\$0
25		1%	\$5,898	0.04	\$177
50		1%	\$5,898	0.02	\$118
100		1%	\$5,898	0.01	\$59
250		1%	\$5,898	0.004	\$35
			AAL		\$389
Battery Energy Storage System (Flood)					
Return Period	Innud. Depth (m)	Vulnerability	Damages + losses	Probability	Expected Loss
5	0.28 0.29 0.32 0.33 0.53 0.78	0%	\$0.00	20%	\$0
10		0%	\$0.00	10%	\$0
25		1%	\$11,956.37	4%	\$359
50		1%	\$11,956.37	2%	\$239
100		1%	\$11,956.37	1%	\$120
250		1%	\$11,956.37	0.4%	\$72
			AAL		\$789
TOTAL AAL			\$1,178		

Table 5.14: AAL for storm surge event for comprehensive strategy (Scenario 1.1 + 2.2)

Solar PV system (SS)					
Return Period	Innud. Depth (m)	Vulnerability	Damages + losses	Probability	Expected Loss
5	0.20	0%	\$0	20%	\$0
10	0.24	0%	\$0	10%	\$0
25	0.28	0%	\$0	4%	\$0
50	0.32	1%	\$5,898	2%	\$59
100	0.36	1%	\$5,898	1%	\$59
250	0.40	1%	\$5,898	0.4%	\$35
			AAL		\$153
Battery Energy Storage System (SS)					
Return Period	Innud. Depth (m)	Vulnerability	Damages + losses	Probability	Expected Loss
5	0.20	0%	\$0.00	20%	\$0
10	0.24	0%	\$0.00	10%	\$0
25	0.28	0%	\$0.00	4%	\$0
50	0.32	1%	\$11,956.37	2%	\$120
100	0.36	1%	\$11,956.37	1%	\$120
250	0.40	1%	\$11,956.37	0.4%	\$72
			AAL		\$239
TOTAL AAL			\$392		

Table 5.15: AAL for erosion event for comprehensive strategy (Scenario 1.1 + 2.2)

Solar PV system (Erosion)					
Return Period	Shoreline retreat (m)	Vulnerability	Damages + losses	Probability	Expected Loss
5	0.00	0%	\$0	20%	\$0
10	0.00	0%	\$0	10%	\$0
25	0.00	0%	\$0	4%	\$0
50	0.00	0%	\$0	2%	\$0
100	10.00	0%	\$0	1%	\$0
250	20.00	0%	\$0	0.4%	\$0
			AAL		\$0
Battery Energy Storage System (Erosion)					
Return Period	Shoreline retreat (m)	Vulnerability	Damages + losses	Probability	Expected Loss
5	0.00	0%	\$0.00	20%	\$0
10	0.00	0%	\$0.00	10%	\$0
25	0.00	0%	\$0.00	4%	\$0
50	0.00	0%	\$0.00	2%	\$0
100	10.00	0%	\$0.00	1%	\$0
250	20.00	0%	\$0.00	0.4%	\$0
			AAL		\$0
TOTAL AAL			\$0		

5.5.2.4 Extended Comprehensive Scenario

The extended comprehensive scenario would have utilized the costliest but the most effective mitigation scenarios for protecting the shoreline and reducing the risk of flood damage to assets. However, this was not considered due to the relatively high cost and low risk of erosion. The solar farm's proposed site area has a buffer of 60m setback from the existing shoreline. With erosion rates on Leguan Island not exceeding 5mm/year, the likelihood of complete site inundation during the lifetime of the project equipment is substantially low.

5.6 Summary

- The hazards considered for the project site were fluvial flooding, storm surge flooding and erosion. The dominant flooding event was that of the river with a 250-year RP water surface level of 0.78m. Shoreline retreat went as far as 20m
- The proposed mitigation strategies against these hazards would raise the site above the expected flood level as well as use hard structures to stabilize the shoreline. These were analysed in the following combinations:
 - Unmitigated strategy (no action)
 - Flood mitigation strategy (Scenario 1.1: Stilts/posts and Plinths)
 - Erosion mitigation strategy (Scenario 2.2: Revet,ent)
 - Comprehensive Strategy (flooding and erosion protection, Scenario 1.1 + Scenario 2.2)

The total cost for each scenario is seen in Table 5-13.

Table 5.16: Summary of the total costs for all mitigation scenarios

	Scenario	Item	Cost (USD)
Flooding	1.1	Stilts/posts and Plinths	\$142,914.98
	1.2	Site Grading and fill	\$399,448.90
Erosion	2.1	Groins	\$4,560,000.00
	2.2	Revetment	\$1,435,406.70

- Damage curves and AAL values for each hazard and corresponding mitigation scenarios were done. With no mitigation, the site remained most vulnerable with inundation causing complete failure for the PV panels at 0.9m and for the BESS at 0.45m. Raising the site above the 1.2m height (above EGL) allowed for damage to occur for a 250-yr RP event. Erosion had no effect on the site for an event with a less than 250-year RP.

Table 5.17: Recommended elevations for assets

Flood Event	Flood Elevation (m)	Recommended Elevation Finished (m)
250 RP Storm Surge	+2.08	+2.1
250 RP Fluvial Flooding	+1.6	
250 RP Pluvial Flooding	+1.0	

- The AAL was highest for the unmitigated scenario, with repairs and replacement of the equipment most probable when no mitigation was implemented. A summary is seen in Table 5-14.

Table 5.18: Summary of AAL for each mitigation scenario

Scenario	AAL (USD)
Unmitigated	\$18,452
Mitigated (with scenario 1: raising asset elevation)	\$2,595
Mitigated (with scenarios 1 and 2: raising asset elevation and shoreline protection)	\$2,595

6 Financial analysis

Financial analysis methods were used to determine the most feasible and reasonable mitigation strategy compared to the unmitigated strategy. A 'baseline' strategy has also been analysed which uses values obtained from the GEA. The mitigation strategies analysed were taken from Section 5.2. This approach was used to determine the cash inflows and normal operating expenditures with the varying expected rates of return for each strategy of action/inaction. It should be noted that an acceptable IRR for solar mini-grids is about 11%¹⁴. Financial analysis was performed by comparing the internal rate of return (IRR) and the cost-benefit ratio (CBR) between all scenarios while accounting for downtime (time lost from equipment unavailability). With the assumption that both the PV and BESS are down for 8 weeks after a hazard event, the cost of shutdown is USD 40,320.00 for each 8-week period of repairs.

The IRR analysis was done for the following:

1. Base scenario (without considering natural hazards as proposed by the GEA)
2. Unmitigated strategy
3. Mitigated strategy
 - a) Flood mitigation Strategy (Scenario 1.1)
 - b) Erosion Mitigation Strategy (Scenario 2.2)
 - c) Comprehensive Mitigation Strategy (Scenario 1.1 + Scenario 2.2)

A key underlying assumption of the analysis is the kokers work adequately. Should any of the kokers fail then the flood levels would increase from 0.78 to 1.12 meters for the 250-year RP.

6.1 Base scenario (GEA)

The base scenario (provided by GEA) will be used as the reference point for the study with which all findings will be compared with. Based on Guyana's Sea Defences (9 ACP), IRR for the project is at 9.7%. Table 8.1 shows a similar IRR value obtained with an assumed social discount rate of 3.1%.

Table 6.1: IRR for base scenario

Costs	USD
Project Cost (USD)	\$1,785,452
Benefits (USD)	\$204,311
Lifespan of project	20
Financial Analysis (USD)	
Net Present Value: Benefits (USD)	\$3,011,718.20
Cost/Investment (USD)	\$1,785,452.00
Internal rate of return	9.6%
Benefit: Cost	1.69

6.2 Unmitigated Strategy

This strategy applies the annualized losses (due to equipment damage and downtime) to the site without any mitigative measure against flooding or erosion. This helps to determine a realistic idea of the feasibility of the project given the current climate of the site. The positive IRR indicates that carrying out the project without mitigation measures

¹⁴ The World Bank. (2018). International Development Association Project Appraisal Document On A Proposed Strategic Climate Fund Scaling Up Renewable Energy Program Grant In The Amount Of Us\$5.61 Million And A Proposed Strategic Climate Fund Scaling Up Renewable Energy Program Credit In The Amount Of Us\$2.0 Million To Nepal For A Nepal: Private Sector-Led Mini-Grid Energy Access Project. [World Bank Document](#)

may be profitable. This scenario without mitigative measures produces the highest amount in terms of deferred losses, but the revenue from renewable energy (RE) is still not enough to offset the Annualized Losses from the hazards within the lifetime of the solar farm.

Table 6.2: IRR for unmitigated strategy

Costs		USD
Project Cost (USD)		\$1,785,452
Average Annual Losses - unmitigated (USD)		\$14,591
Benefits (USD)		\$185,859
Lifespan of project		20
Financial Analysis (USD)		
Net Present Value: Benefits (USD)		\$2,796,629.03
Cost/Investment (USD)		\$1,785,452.00
Internal rate of return		8.6%
Benefit: Cost		1.57

6.3 Mitigated Scenarios

The mitigated strategies compare the various mitigative measures proposed and the deferred losses that can be obtained from each scenario to determine the most feasible mitigation measure applicable to the project. The combination of scenarios remains the same for the 2 strategies:

1. Flood mitigation strategy (Scenario 1.1)
2. Erosion mitigation strategy (Scenario 2.2)
3. Comprehensive Strategy (Scenario 1.1 + Scenario 2.2)

6.3.1 Flood mitigation strategy

This strategy applies the annualized losses (due to equipment damage and downtime) to the site with the mitigative measure: raising assets using stilts/posts and plinths. A summary is seen in Table 6.3. The adaptation cost for raising the assets is now applied to the project, increasing the implementation cost. However, with the new protective measure, the deferred loss is increased, raising the benefits of carrying out the project.

Table 6.3: IRR for Flood mitigation strategy

Costs		USD
Project Cost (USD)		\$1,785,452
Adaption Cost (USD)		\$142,915
Total implementation Cost (USD)		\$1,928,367
Average Annual Losses - mitigated (USD)		\$1,571
Benefits (USD)		\$202,740
Lifespan of project		20
Financial Analysis		
Net Present Value: Benefits (USD)		\$2,988,562.12
Cost/Investment (USD)		\$1,928,366.98
Internal rate of return		8.43%
Benefit: Cost		1.55

6.3.2 Erosion mitigation strategy

This strategy applies the annualized losses (due to equipment damage and downtime) to the site with the mitigative measure: constructing a revetment. A summary is seen in Table 6.4. The adaptation cost for filling the site is now applied to the project, increasing the implementation cost. With the new protective measure, the deferred loss is reduced to zero since the site is not expected to be significantly affected by erosion.

Table 6.4: IRR for erosion mitigation strategy

Costs		USD
Project Cost (USD)		\$1,785,452
Adaption Cost (USD)		\$1,435,407
Total implementation Cost (USD)		\$3,220,859
Average Annual Losses - mitigated (USD)		\$14,591

Benefits (USD)	\$202,431
Lifespan of project	20
Financial Analysis	
Net Present Value: Benefits (USD)	\$2,796,629.03
Cost/Investment (USD)	\$3,220,858.70
Internal rate of return	1.6%
Benefit: Cost	0.87

6.3.3 Comprehensive strategy

This strategy applies the annualized losses (due to equipment damage and downtime) to the site with the mitigative measure: stilts/posts and plinths along with a revetment. A summary is seen in Table 6.4. The adaptation cost for both measures are now applied to the project, further increasing the implementation cost. This implementation cost sees most of its benefit from flood mitigation since erosion has little effect on the site. Even so, the cost for protection against all the hazards is too high given the benefit of the solar farm, indicating that defending against both flood water and erosion is not feasible and requires a more cost-effective alternative. This also highlights that the cost to implement shoreline protection should not be placed on the GEA as it will make the project not feasible. Even doubling the size of the project to within 30m of the proposed sea defence still gives an IRR of 3.4 % making it still infeasible.

Table 6.5: IRR for a comprehensive strategy

Costs	USD
Project Cost (USD)	\$1,785,452
Adaption Cost (USD)	\$1,834,856
Total implementation Cost (USD)	\$3,620,308
Average Annual Losses - mitigated (USD)	\$1,571
Benefits (USD)	\$202,740
Lifespan of project	20
Financial Analysis (USD)	
Net Present Value: Benefits (USD)	\$2,988,562.12
Cost/Investment (USD)	\$3,620,307.60
Internal rate of return	1.1%
Benefit: Cost	0.83

6.4 Summary

Preliminary financial analysis indicates that the development of the solar farm on the Leguan shoreline is viable, albeit not attractive in terms of returns. With all considerations made, the mitigative measures to combat damage from the fluvial flood adequately outweigh the Renewable Energy (RE) savings and deferred losses for the 20-year lifespan of the project. However, when introducing erosion mitigation, the project becomes less feasible. Erosion mitigation measures will have to be viewed as a social contribution and external to the project in order for the project to be deemed viable. The summary of the financial analysis for the project site has highlighted that been prepared in Table 6.5.

Table 6.6: Summary of financial analysis

Costs	Baseline	Unmitigated	Flood mitigation strategy	Erosion mitigation strategy	Comprehensive (Flooding + Erosion) Strategy
Project Cost (USD)	\$1,785,452	\$1,785,452	\$1,785,452	\$1,785,452	\$1,785,452
Adaption Cost (USD)	\$0	\$0	\$142,915	\$1,435,407	\$1,578,322
Total implementation Cost (USD)	\$1,785,452	\$1,785,452	\$1,928,367	\$3,220,859	\$3,363,774
Average Annual Losses (USD)	\$0	\$14,591	\$1,571	\$14,591	\$1,571
Benefits (USD)	\$189,720	\$189,720	\$202,740	\$189,720	\$202,740
Financial Analysis					
Net Present Value: Benefits (USD)	\$3,011,718	\$2,796,629	\$2,988,562	\$2,796,629	\$2,988,562
Cost/Investment (USD)	\$1,785,452	\$1,785,452	\$1,928,367	\$3,220,859	\$3,363,774
Internal rate of return	9.6%	8.6%	8.4%	1.6%	1.8%
Benefit: Cost	1.69	1.57	1.55	0.87	0.89

7 Conclusion and Recommendations

An assessment of the flood plain, wave climate and storm surge indicate that the project site is susceptible to inundation because of ponding and storm surge. The impacts of such however should be mitigated by the presence of tidal control structures in the form of kokers. Under normal operating conditions it is expected that inundation on site will be driven by typical pluvial events, wherein there is ponding due to the closure of seaward outlets. The estimated depths were estimated to be within a range of 0.2 to 0.8m in a 250 Yr. precipitation event. It is recommended that electrical equipment be protected by either site filling, or extended posts/solar racking stilts and platforms to the BESS, transformer, and other electrical systems.

Extremal analysis was conducted to determine the offshore wave heights for the 12-hour swell wave heights and wave heights for the hurricane Scenarios. The analysis deduced that the site would be partially inundated by the storm surge under 5 to 250 Yr Return Period events and would cause damage. The minimum recommended elevation of equipment should be above the 250-year flood elevation, with SLR and above the MSL, of +2.1m elevation, as such should be used as the minimum equipment base elevation.

The project site is susceptible to short-term erosion from storms, with horizontal erosion predicted to range from 8m to 20m, for the 5 and 100 RP storms respectively. It is important to note that occurrences of storms in the area are rare, and as such, the extent of erosion will only be experienced during a worst-case scenario where a hurricane travels close to the mainland. Shoreline protection is recommended to secure the project area.

Several development scenarios were analyzed ranging from an unmitigated strategy to a comprehensive strategy. Each strategy considered the cost of shutting down the plant for repairs and replacement of damaged equipment, with the assumption of that the plant's downtime would be for 8 weeks and lose \$40,320.00 in that time. After comparisons were made, raising the assets' elevations using stilts/posts and plinths was the most effective mitigation strategy against flooding. For the erosion hazard, the use of revetments was also the more cost-effective method for stabilizing the shoreline, with the added benefit of keeping within common local engineering practices. The Leguan solar farm is feasible when using flood mitigation measures without erosion mitigation and is the recommended approach.

Table 7.1: Summary of financial analysis

Costs	Baseline	Unmitigated	Flood mitigation strategy	Erosion mitigation strategy	Comprehensive Strategy
Project Cost (USD)	\$1,785,452	\$1,785,452	\$1,785,452	\$1,785,452	\$1,785,452
Adaption Cost (USD)	\$0	\$0	\$142,915	\$1,435,407	\$1,578,322
Total implementation Cost (USD)	\$1,785,452	\$1,785,452	\$1,928,367	\$3,220,859	\$3,363,774
Average Annual Losses - mitigated (USD)	\$0	\$0	\$1,571	\$14,591	\$1,571
Benefits (USD)	\$189,720	\$189,720	\$202,740	\$189,719.63	\$202,740
Financial Analysis					
Net Present Value: Benefits (USD)	\$3,011,718	\$2,796,629	\$2,988,562	\$2,796,629	\$2,988,562
Cost/Investment (USD)	\$1,785,452	\$1,785,452	\$1,928,367	\$3,220,859	\$3,363,774
Internal rate of return	9.6%	8.6%	8.4%	1.6%	1.8%
Benefit: Cost	1.69	1.57	1.55	0.87	0.89

The recommendations to execute this project are as follows:

- Critical infrastructure should be placed at a Minimum elevation of +2.1m AMSL with the use of silts for the PV panels and plinths for the BESS.
- The construction of the shoreline protection revetment, proposed by the Sea Defense Board, should be a precondition to the construction of the solar farm. Additionally, the revetment structure should be able to withstand a 50 RP event.
- A condition assessment of the kokers in the project area is recommended for further risk management. Such an assessment should also include a drainage assessment of the project area, and the island at large for a 50-year RP event, is also recommended for further risk management.

8 Appendices

8.1 Median Grain Size (D_{50})

The median grain size for the samples ranged from 0.45mm (medium sand) to 0.885 mm (coarse sand) resulting in an average grain size of 0.656 mm (Coarse sand) for all samples collected. The general trend observed of the sediment samples collected is the samples collected further offshore are coarser to about 1m and the samples collected at the berm are finer. This can be observed in Figure 2.8 and can be attributed to the wave climate indicating that 1m is the beginning of the surf zone washing out the fines further offshore.

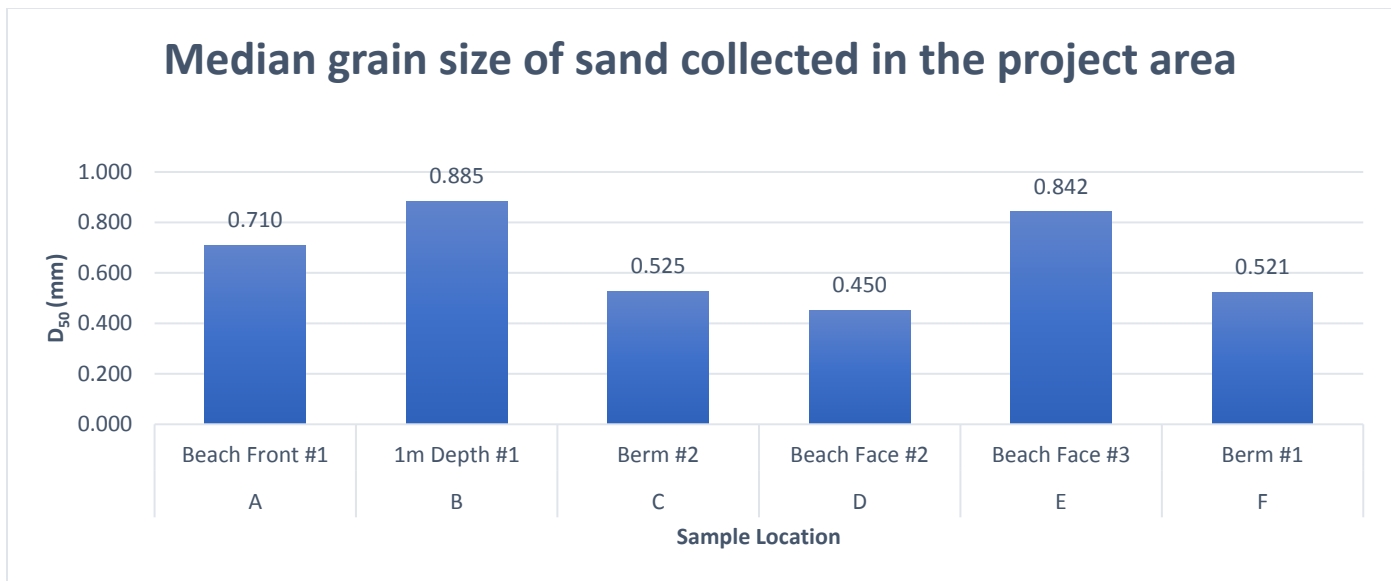


Figure 8.1: Median grain size of sand collected in the project area

8.2 Uniformity Coefficient

The uniformity coefficient is a measure of the variation in particle sizes. It is defined as the ratio of the size of the particle that has 60 percent of the material finer than itself, to the size of the particle that has 10 percent finer than itself.

Within the unified classification system, the sand is well-graded if U_c is greater than or equal to 6. All the samples analyzed had a uniformity coefficient of less than 6 indicating that the sands are not well-graded, as shown in Figure 2.9. This indicates the wave climate arriving at the shoreline ranges from moderate to aggressive causing the finer particle to be washed out leaving the sediments poorly graded.

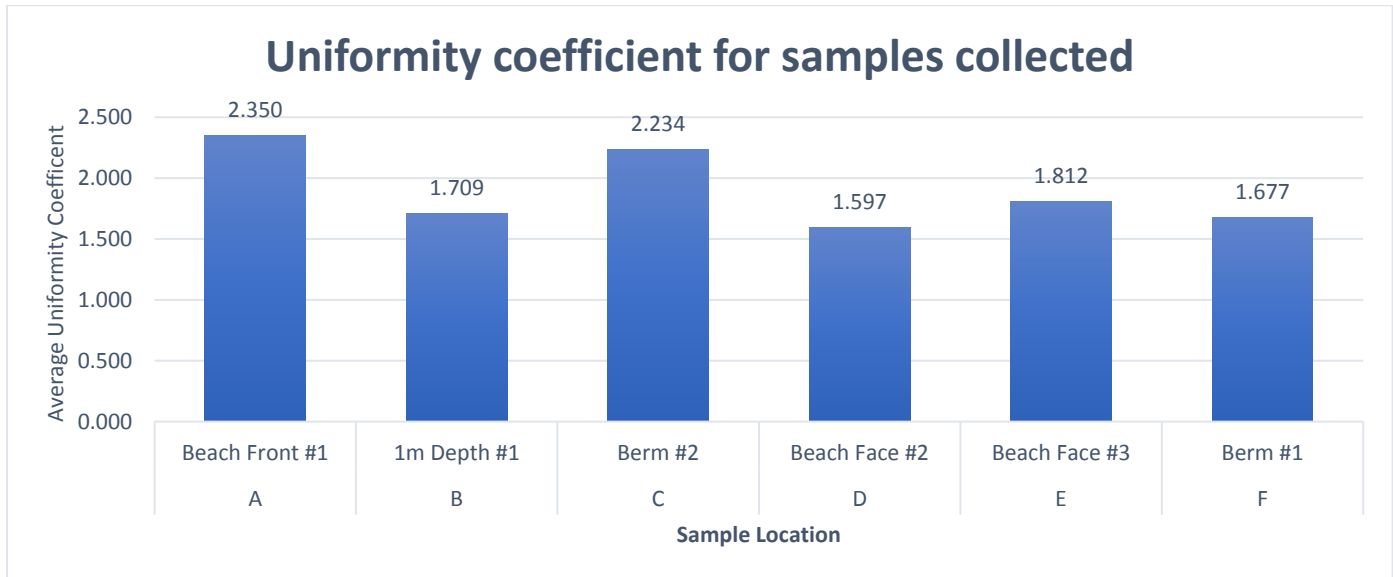


Figure 8.2: Uniformity coefficient for samples collected along the project area shoreline

8.3 Skewness

Skewness describes the shift in the distribution of the normal. The results for skewness for the shoreline show that Profiles C, D and F are strongly positively skewed (> 1.000), profiles A and E are positively skewed and Profile B is near-symmetrical (≈ 0.500), See Figure 2.10. This is indicative of a long coarse tail of particles and a moderate to aggressive wave climate that washes out the finest particles in profiles C, D and F.

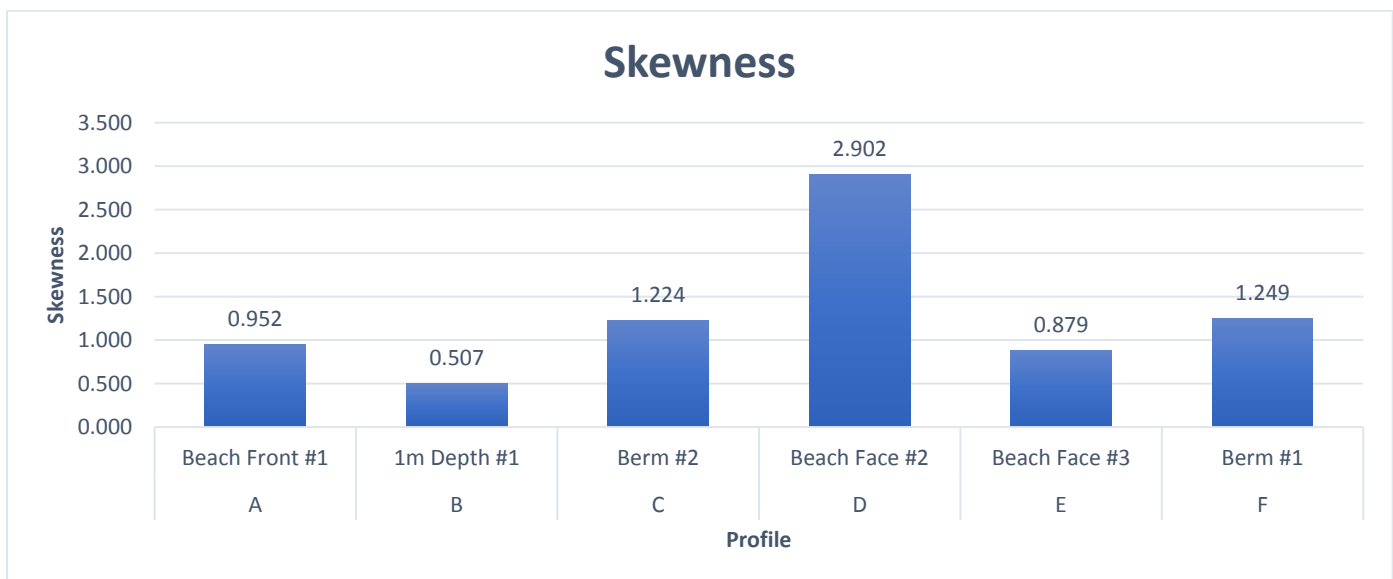


Figure 8.3: Skewness for samples collected along the project area shoreline

8.4 Kurtosis

Kurtosis was determined to be within the range of 0.798 and 1.413 for all samples collected. This indicates that the beach sediments range from platykurtic to leptokurtic. This is indicative of moderate to aggressive coastal processes (sediment transport) that sort out the particles into a discrete particle size. See Figure 2.11.

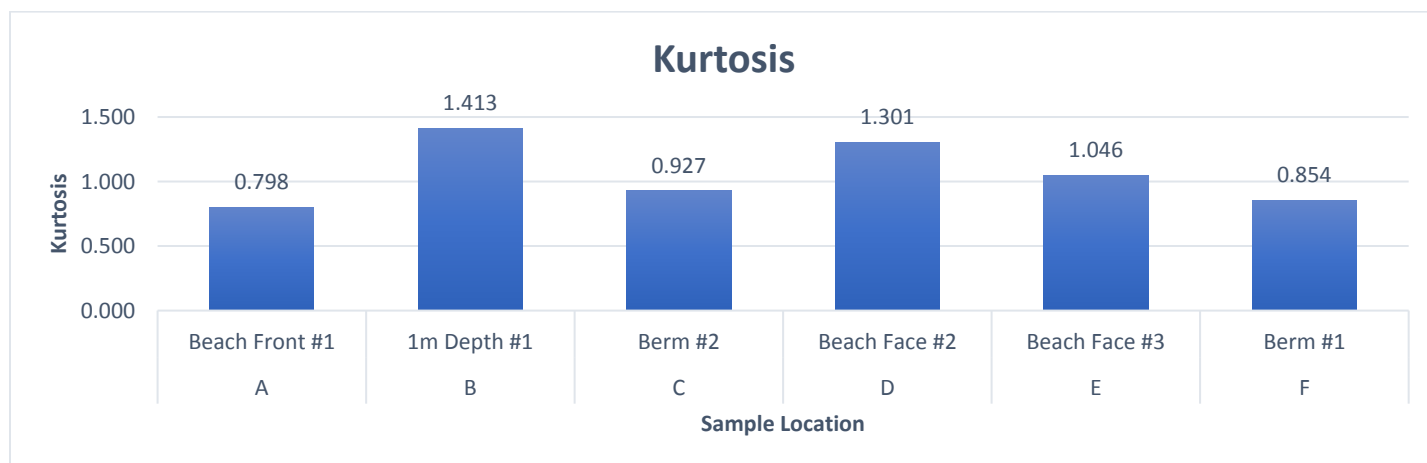


Figure 8.4. Kurtosis for samples collected along the project area shoreline

8.5 Downtime calculation

Table 8.1: downtime calculation for solar farm

Components	Units	PV	BESS
Value	USD	\$589,814.86	\$1,195,637.14
Ground elevation	m	1.9	1.9
Height of vulnerable port above plinth level	m	0	0
Climate Change and Project life			
Sea Level rise rate	mm/year	6	6
Project life	year	20	20
sea level rise	m	0.12	0.12
Adverse Consequences of shut down			
GEA price of generation	USD per KWH	\$0.18	\$0.18
Consumer price	USD per KWH	\$0.32	\$0.32
Spread price of generation	USD per KWH	\$0.14	\$0.14
	Utilization	90%	90%
Generation rate	Mwh/d	2.5	2.5
Length of shut down	Weeks	8	8

Cost of shut down	USD	\$40,320.00	\$40,320.00
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